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UTILIZATION AND INTERPRETATION OF HYDROLOGIC DATA WITH SELECTED EXAMPLES FROM NEW HAMPSHIRE

by

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COMPLETION REPORT

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Chapter I

INTRODUCTION

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PURPOSE AND SCOPE

The modern emphasis on comprehensive planning and environmental studies has created an increasing demand for the interpretation and utilization of hydrologic data. Not only is there an emphasis on such studies, but in fact there may be legal requirements as well. A potential user of hydrologic data is faced with at least three kinds of problems: 1) Availability of data and adequacy of data collection programs; 2) What to do with the data that are available; and 3) How to transfer available data from a collection site to an unmeasured location of interest.

A lack of data and the problem of inadequate data collection programs are beyond the scope of this report, which instead is focused on interpretation and utilization of what are available. The intent is to show mainly by examples some basic things that can be done. The examples are drawn from streamflow records because these probably are of primary, although not sole, concern for the intended audience which is that broad group of professional workers who do not have strong backgrounds in hydrology.

The general plan of the report is to give examples of various ways of manipulating and interpreting streamflow data. First, there is a discussion of data sources and useful works available in the hydrology literature. This preliminary material is followed by Chapter II, which deals with the basic elements of frequency analysis. Chapter III is a

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more detailed description of how to synthesize flow-duration curves for ungaged areas. This is followed by Chapter IV which has an application of a regional flood analysis, mainly by graphical methods. The report is concluded by Chapter V which gives selected examples of the use of regression analysis.

SOURCES OF INFORMATION

The major source of information for streamflow data is the records of the U.S. Geological Survey which maintains the basic stream-gaging network of the United States. The records may be consulted at the various U.S.G.S. Offices (Boston, MA, and Concord, NH, for New Hampshire) and, in addition, they are published on a regular basis in the U.S.G.S. Water-Supply Papers and interim reports. For New Hampshire, the basic reference is Water Resources Data for New Hampshire and Vermont, U.S. Geological Survey, Water-Data Report NH-VT-76-1, and the various reports cited therein. It should be noted that these publications also contain information on water quality and groundwater levels.

Various State and Federal agencies maintain streamflow and other data collection programs for specific purposes. These data are not commonly published, but probably can be obtained from the agency. The problem currently is to find out about them. Precipitation and other climatologic data are collected and published by the National Weather Service (formerly the U.S. Weather Bureau) in the monthly weather summaries. The records may be consulted at the various National Weather Service Offices (Concord, NH, for New Hampshire).

The published records mentioned in this section may also be viewed or obtained in New Hampshire at libraries of the University of New Hampshire System, the State Library in Concord, and the Water Resource Research Center at UNH in Durham.

SELECTED REFERENCES

Important references will be cited at appropriate places in the text, and they will be listed in a "References Cited" section at the end of this report. It seems useful at this point, however, to list some more general works that may be of use to the reader. Most of these works should be available at some of the libraries mentioned in the prior section and in particular at UNH in Durham.

GENERAL HYDROLOGY

Handbook of Applied Hydrology. Edited by Ven Te Chow. McGraw-Hill Company. 1964.

Handbook on the Principles of Hydrology. Edited by D. M. Gray. Water Information Center, Inc. 1973.

AGENCY PROCEDURES

Hydrologic Engineering Methods for Water Resource Development. Multi-volume set by the Hydrologic Engineering Center, U.S. Army Corps of Engineers (distributed by the National Technical Information Service).

National Engineering Handbook. Multi-chapter set by the Soil Conservation Service, in particular Section 4. Hydrology (distributed by the Superintendent of Documents).

Techniques of Water-Resources Investigations of the United States Geological Survey. Multi-book and -chapter set (distributed by the Geological Survey).

Water-Supply Papers. Multi-volume set (distributed by the Geological Survey).

HYDROLOGY TEXTS

Applied Hydrology. R. K. Linsley, Jr., and others. McGraw-Hill Company. 1949.

Engineering Hydrology. S. S. Butler. Prentice-Hall. 1957.

Hydrology. C. O. Wisler and E. F. Brater. John Wiley and Sons, Inc. 1949.

Hydrology for Engineers. R. K. Linsley, Jr., and others. Second edition. McGraw-Hill Company. 1975.

Introduction to Hydrology. Warren Viessman, Jr., and others. Second edition. IEPA Dun-Donnelley Publisher. 1977.

Principles of Hydrology. R. C. Ward. Second edition. McGraw-Hill Company. 1975.

Water Resources Engineering. R. K. Linsley, Jr., and J. B. Franzini. Second edition. McGraw-Hill Company. 1972.

Water in Environmental Planning. T. Dunne and L. B. Leopold. W. H. Freeman Company. 1978.

Chapter II

BASIC DATA AND METHODS OF PRESENTATION

Francis R. Hall

INTRODUCTION

The basic data to be considered herein are taken from streamflow records for New Hampshire (U.S. Geological Survey, 1977, and reports referenced therein). The streamflow unit is the average daily discharge in cubic feet per second (cfs). Other units or other time periods may be used for special purposes. Daily discharge values are published for a "water year" which begins October 1 and ends September 30 of the following year, and which is referred to by the year in which it ends. The water year is considered to be somewhat more of a natural hydrologic year than is the calendar year which splits the winter.

The daily discharge represents the volume of water or cfs-day that passes the stream gage during a 24-hour period. Actual or instantaneous discharge at any specified time may be greater or smaller than the daily discharge, which represents an average flow for 24 hours. One cfs is equal to 7.48 gallons per second, 448.8 gallons per minute, or 28.32 liters per second. Also, one cfs for one day (cfs-day) is equal to 86,400 cubic feet, 646,000 gallons, or 2.45 X 10^6 liters. On an areal basis, one cfs-day is equivalent to 0.0372 inches runoff from one square mile or 2.45 millimeters from one square kilometer.

An obvious difficulty when utilizing daily discharges for some period of record is the sheer volume of data. For example, a record of 10 years (about the shortest that can be reliably worked with) has slightly more than 3650 daily values depending on the number of leap years included. The traditional method which is the one to be followed herein is to assume the data are independent and randomly drawn from a streamflow population, and therefore, to assume that a probablistic (time-independent) approach can be used. Two problems arise, however, in that the underlying probability distribution is unknown and that the data are actually time-dependent (stochastic). In fact, the data form a time series with daily discharges being fairly highly correlated with prior daily discharges, weekly discharges being less correlated, monthly even less, and so on to where in most New Hampshire streams under natural conditions there is little correlation after a few years.

Time-series analysis of hydrologic data represents a level of complexity that is beyond the scope of this report. Therefore, the assumption will be made that a time-dependent (probabilistic) approach will produce satisfactory results which can be interpreted in terms of expectations for given frequencies but which cannot provide information about when something actually will occur (Chow, 1964). Also, some inferences and assumptions are made where necessary about possible probability distributions. As will be discussed later, daily streamflows do not usually follow a normal or gaussian (bell-shaped) distribution (Riggs, 1968). They are generally skewed with the average discharge being considerably larger than the median discharge. The data are characterized by extreme values at high and low discharges. The logarithms of the daily discharges may more closely approach a normal distribution; so this is a commonly used transformation. Finally, there is a tendency toward normality as the flow period increases. That is, annual discharges are more nearly normally distributed than are daily discharges.

If a stream is regulated in some fashion, then there is human interference which complicates matters even further. Therefore, this report will be concerned mainly with unregulated (natural) streams.

FLOW DURATION

One of the more useful ways of summarizing daily discharge values for a period of record is by a type of frequency analysis referred to as flow-duration (Searcy, 1959). Normally, the results are presented in the form of flow-duration curves. The discharges are arranged by magnitude without regard to time of occurrence, and the frequencies are usually cumulated from highest to lowest discharge. In this way, the results can be interpreted as percent of time a given flow is equaled or exceeded (Figure 1). Chapter III includes a discussion of interpretation of flow-duration curves; so the following material will be concerned mainly with the practical details of obtaining them from stream gaging records.

The magnitude-frequency array discussed above is commonly done by class intervals because of the number of discharge values. The analysis can be done manually but is tedious; therefore, the data are generally processed by digital computer. Uniform class intervals are not very effective, however, because of extreme events at high and low discharges, which cause the discharge to range over two or more orders of magnitude. Usually the intervals are selected by a logarithmic progression so as to provide about 30-35 fairly evenly spaced data points (Searcy, 1959). This also means that plotting discharge on a uniform or arithmetic scale is not very effective, and a common practice is to plot logarithms of the discharges or it is easier to use semi-logarithmic graph paper as is done on Figure 1.

Another consequence of the data spread is that a uniformly spaced frequency plot does not do justice to the high and low discharges. Therefore, the tendency is to use a normal probability scale as is done in Figure 1. The skew referred to earlier in this chapter is shown by the spread between the mean and median on Figure 1. Some other implications are that the daily discharges for the Lamprey River are not normally distributed, but the logarithms of discharge are more or less normal except at low and high flows. That is, the points should plot a straight line or reasonably so on lognormal paper if they are lognormally distributed. Other streams in New Hampshire show patterns similar to the Lamprey (see Chapter III). If the time interval is extended to weekly, monthly, or annual discharges, then the flow-duration curve tends to become flatter and to approach a straight line as the time period increases.

A flow-duration curve displays the characteristics of a stream as recorded at a specific location. Therefore, drainage-basin area and other hydrologic characteristics must be considered if one wishes to compare areas of different sizes or if one seeks to estimate flow-duration curves for ungaged areas (see Chapter III). Two common ways of alleviating these problems are to plot the streamflow per unit area, or to plot streamflow as a ratio to some characteristic value. The first approach is illustrated in Figure 2 which shows flow-duration curves for some streams in southeastern New Hampshire plotted as cfs/square mile or CFSM. The same thing

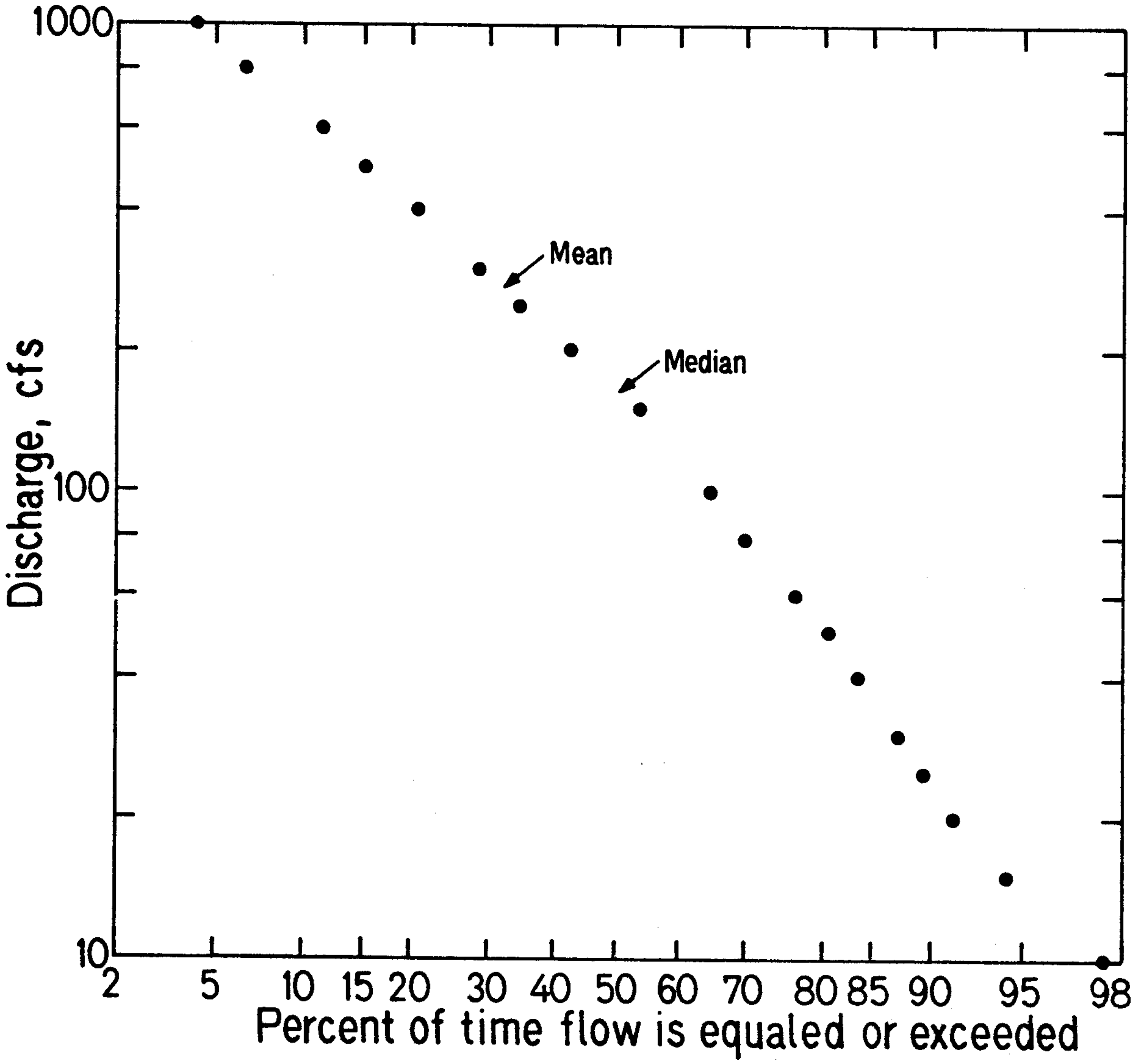

Figure 1. Flow-Duration Curve for Lamprey River Near Newmarket, New Hampshire, 1935-1972.

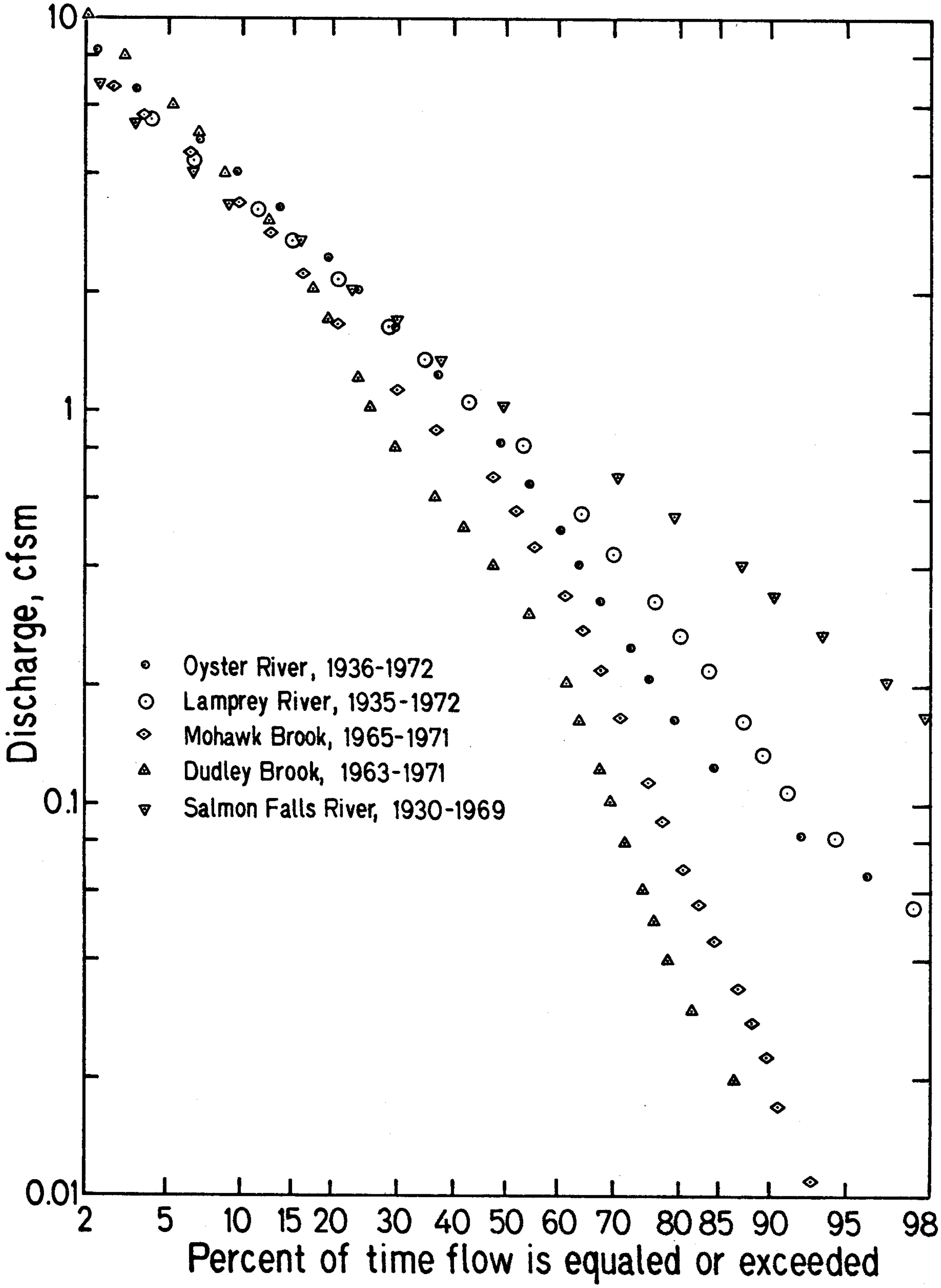

Figure 2. Flow-Duration Curves for Selected Streams.

could be done with other units such as area inches (either acres or square miles) or area millimeters (either hectares or square kilometers). Chapter III has illustrations of the second approach where the characteristic value is mean annual discharge.

It is of interest to note that on Figure 2, the Salmon Falls River is considerably regulated by ponds and the Lamprey River is somewhat regulated by ponds, whereas the other three streams are unregulated. The more obvious effects of regulation are that the curves for the Salmon Falls and Lamprey as compared to the other three streams tend to be flatter with lower discharges per square mile at higher flows and with higher discharges per square mile at lower flows. This is to be expected as regulation by ponds tends to smooth out the pattern of flow. The low-flow end of the Oyster River curve also flattens, but as discussed in the next section this is probably due to groundwater inflow from the Lamprey basin.

LOW FLOWS

The flow-duration curve displays the overall flow characteristics of a given drainage basin, but for many purposes it is also desirable to examine more closely the extreme values at high and low flows. A common way this is done is to determine for each year the lowest or highest mean daily discharge for selected numbers of consecutive days (usually 1, 3, 7, 14, 30, 60, 90, 120, 150, 183, and 274). The results also can be treated as flow volumes. Manual tabulation is tedious; so a digital computer is preferred. This section will be concerned with low flows and the next section with high flows.

The water year as previously defined is not satisfactory for low flows because the typical fall recession period is split. Therefore, a low-flow year is defined as beginning April 1, and ending March 31, of the following year. In this case, it is called by the year in which it begins. The values for each year are arrayed from lowest to highest in magnitude. If similar values occur in more than one year, they are still arrayed individually. Plotting positions or probabilities are assigned according to

$$P = \frac{n}{m + 1} \tag{1}$$

where n = 1, 2, 3, 4, . . . m, order of magnitude or rank

m = length of period of record in years

Other plotting position equations are available (Chow, 1964). The results then are interpreted as probability of exceedance. For example, the lowest magnitude event in a 10-year record has a 1/11 (=0.091) probability of being exceeded in any year.

Because the low-flow events are drawn from annual sequences, the probabilities can be looked at in another way by taking reciprocals or calculating the inverse of equation (1) according to

$$T = \frac{1}{P} = \frac{m + 1}{n} \tag{2}$$

where T = return period in years.

The lowest magnitude event in a 10-year record has a return period of 11 years or it can be expected to be exceeded once every 11 years.

Selected low-flow curves and the annual curve for the Oyster River near Durham are displayed on lognormal paper in Figure 3. The abscissa is given in probabilities, but some return periods are given for reference. These low-flow curves are neither normally nor lognormally distributed, although they are fairly close to the latter. Chow (1964) discusses other possible distributions. Figure 4 illustrates an alternative way of looking at the same information for a selected return period. For certain purposes it may be desirable to use discharge per square mile, ratio to a characteristic discharge, and so on, but examples are not included herein.

Before discussing some possible uses for low-flow data as arrayed on Figures 3 and 4, it seems worthwhile to comment briefly on the shapes of the curves on Figure 3. For one thing, the pronounced tendency to flatten out at higher return periods (lower probabilities) is not characteristic of most New Hampshire streams which instead tend to steepen as might be expected. That is, a drainage basin should go dry if it does not rain for a sufficiently long time period. The flattening, on the other hand, suggests that the Oyster River will never go dry. Since this would seem a physical impossibility, a likely explanation is that the Oyster River receives groundwater inflow from the adjacent Lamprey basin. The tendency to "turn up" at lower return periods (higher probabilities) is characteristic of most streams in New Hampshire and reflects both the shortness of record and the effect of extreme events. That is, as the record lengthens the curves will tend to smooth out, but the upward trends will likely still be there on this type of a plot. There are some so-called extremal graphical methods that have been developed in an attempt to straighten out low-flow curves (Chow, 1964).

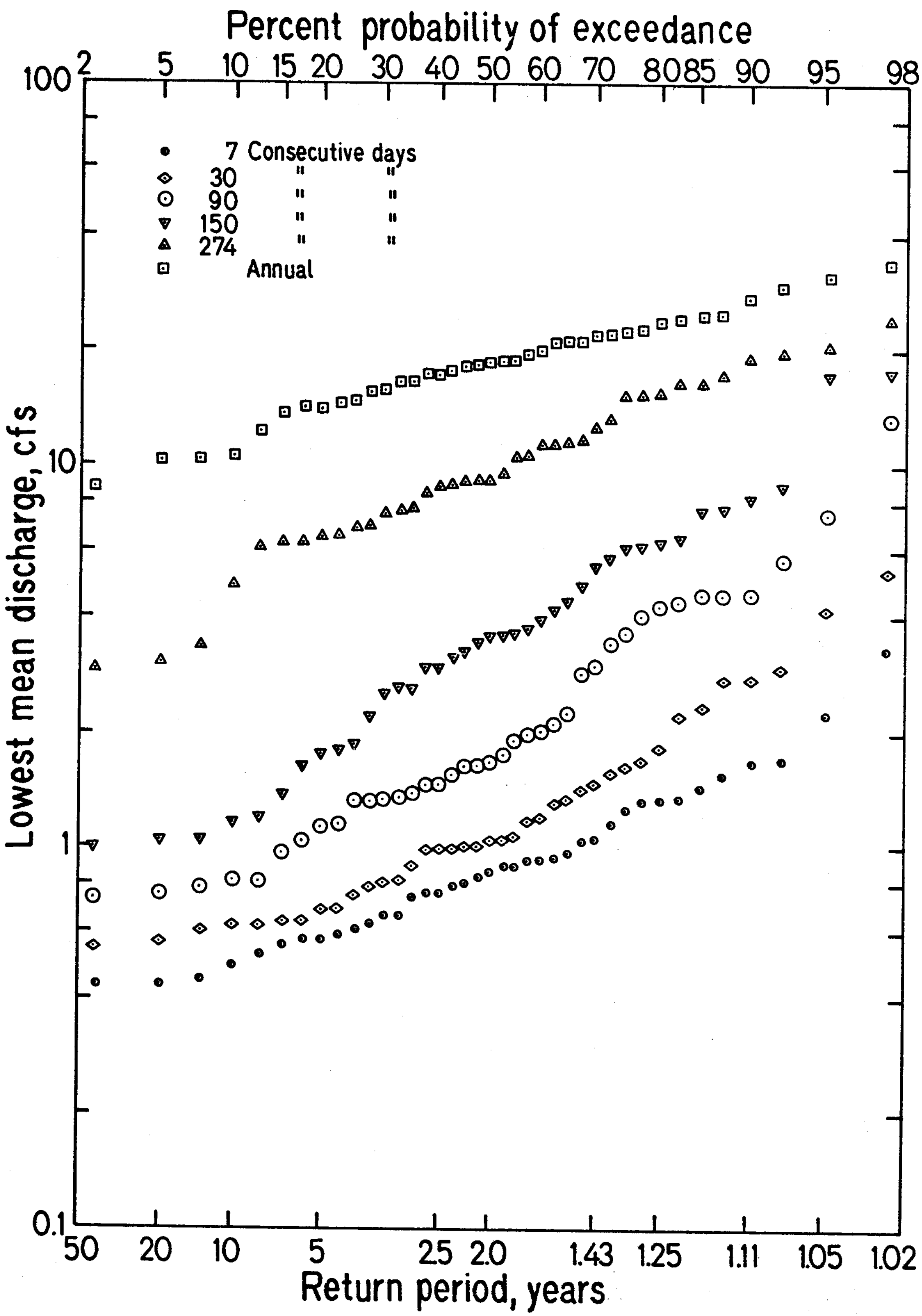

Figure 3. Selected Low-Flow Curves for Oyster River Near Durham, for Low-Flow Year Beginning April 1 for 1936-1974.

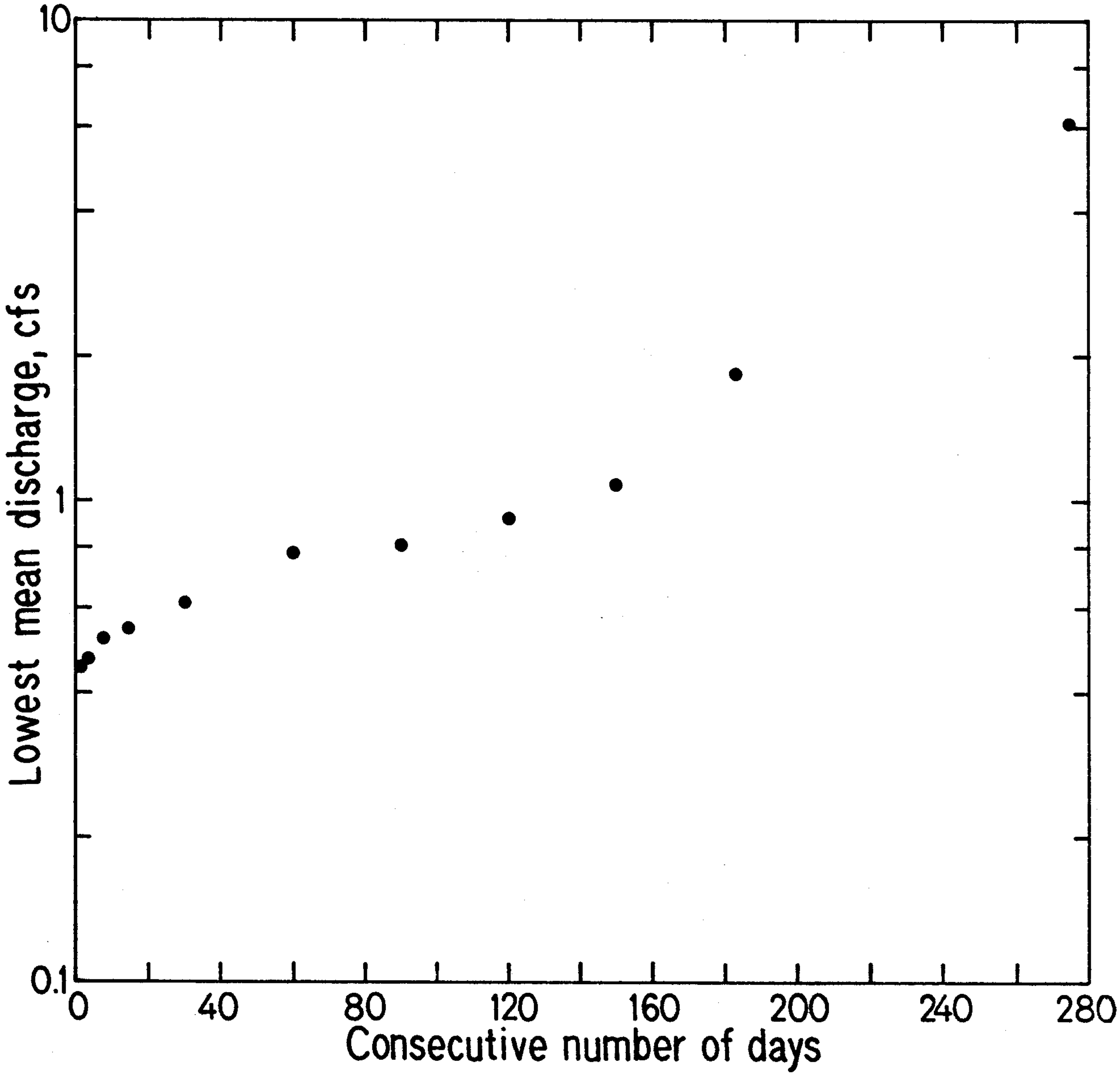

Figure 4. 10-Year Low-Flow Curve for the Oyster River.

As an example of the use of low-flow data, suppose it is desired to take one cfs continuously from the Oyster River. Figure 3 shows that flow will be less than one cfs for the following number of consecutive days for stipulated return periods:

Consecutive Days	Return Period, years
7	1.5
30	2.5
90	6.5
150	20.0

On the average, the flow requirement cannot be met for seven consecutive days every year and a half, for 30 consecutive days every two and a half years, and so on. Therefore, either a risk must be taken or adequate storage must be provided. If a decision is made that flow of less than one cfs for seven consecutive days every 10 years is acceptable, then Figure 3 shows that sufficient storage must be provided for supplemental water to bring flow up to one cfs for about 130 days.

HIGH FLOWS

The high-flow analysis is handled in a fashion similar to low-flows, except the regular water year is used and the data are arrayed from highest to lowest in magnitude. Plotting positions or return periods are assigned such that the highest magnitude event in a 10-year record has a 1/11 (=0.091) probability of occurring in any year or it can be expected to occur once every 11 years.

Selected high-flow curves for the Oyster River near Durham are displayed on lognormal paper in Figure 5. These curves are nearly lognormally

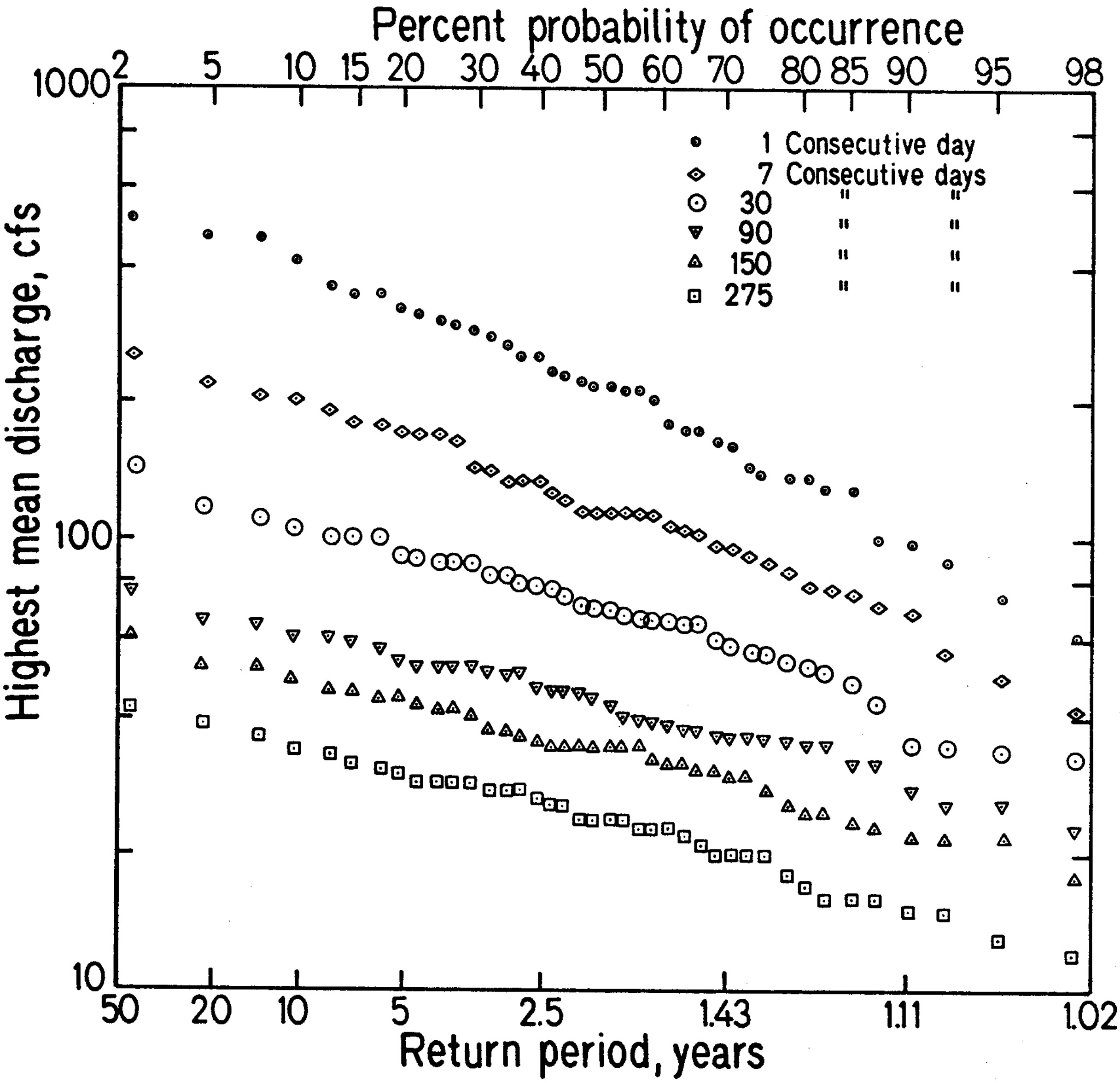

Figure 5. Selected High-Flow Curves for Oyster River.

distributed except for some tendency to turn downward at low return periods (high probability). Such a tendency probably can be attributed to the relatively short period of record. A 10-year high-flow curve is given on Figure 6.

As an example of the use of high-flow curves, suppose that structures on a flood plain can withstand brief flooding on the order of one day but will undergo damage during flooding on the order of two or more days. Also, flooding begins at a flow of 100 cfs. Figure 5 shows the following number of consecutive days for stipulated return periods at 100 cfs:

Consecutive Days	Return Period, Years
1	1.2
7	1.4
30	6.0

Clearly, flooding for more than one day is likely to occur every few years. Therefore, a risk has to be taken or some sort of preventative measures must be provided for.

INSTANTANEOUS DISCHARGES

So far the discussion has been concerned with various manipulations of daily mean discharges. Now, brief consideration is given to the instantaneous or actual discharge at a given instant in time. Such information is available from the stream-gage recorder chart or special crest-height gages. Also, the U.S. Geological Survey reports for its gaging stations the annual high and low instantaneous discharges as well as all peak flows above a selected base level. The major use of these data is for flood flow

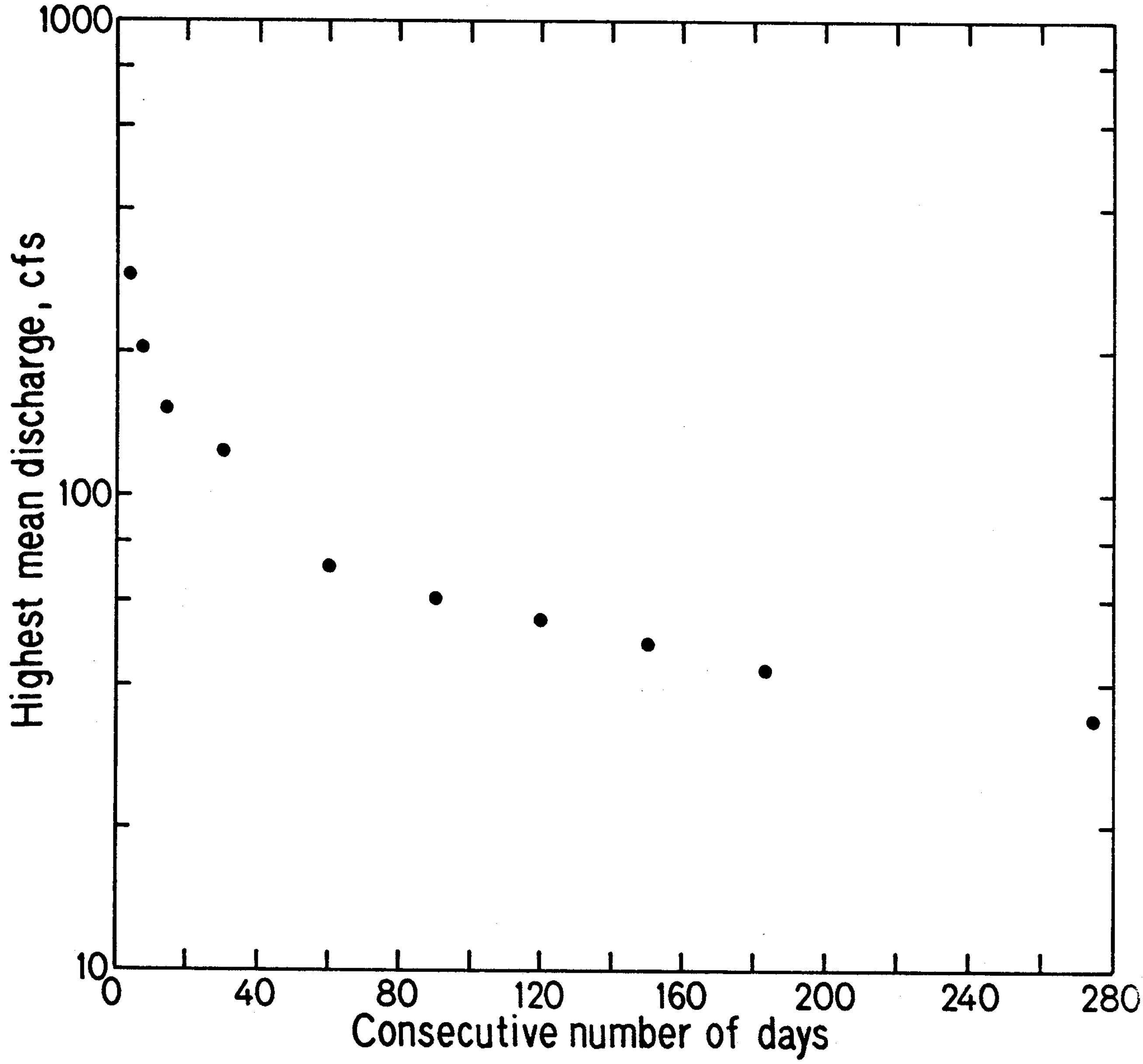

Figure 6. 10-Year High-Flow Curve for Oyster River.

analysis. Generally, the mean one, three, or seven consecutive day flows, as already discussed, are preferred for low-flow or drought studies.

The most common type of flood-flow analysis is begun by arraying the annual instantaneous peaks from highest to lowest magnitude and assigning plotting positions or return periods in a manner comparable to that already discussed for high-flows. If only low return period (high probability) events are of interest, then a graphical plot on lognormal paper will suffice. In most cases, however, major floods are of greatest interest, and available stream records usually are too short to define them directly. Therefore, it is necessary to extend or project the record. The two general ways this is done are by regionalization, either by graphical or regression methods, and by fitting to a probability distribution. The former approach is discussed in Chapters IV and V, and the latter is discussed below.

An alternative approach is to make an analysis as discussed above except that all instantaneous peaks above a predetermined level are utilized. In this case, some years may not be represented at all and other years may be represented by one, two, or more peaks. This is referred to as a partial duration series. The partial series is identical with the annual series at return periods greater than 10 years (Haan, 1977).

The underlying probability distribution for hydrologic data is rarely if ever known; therefore, a distribution must be assumed and a best fit of data attempted. There are numerous possibilities for flood flows, but a discussion is beyond the scope of this report. The interested reader is referred to the various references listed in Chapter I, and in particular to Chow (1964) and Viessman and others (1977). One distribution, the so-called log-Pearson, or more correctly log-Pearson Type III, is worth mentioning, however, because a Federal Interagency Committee has adopted it for nationwide use (WRC, 1976) and because it is the prescribed method for the Flood Insurance Act administered by the U.S. Department of Housing and Urban Development.

Chow (1964) has shown that many probability distributions of interest in hydrology can be represented by a general equation for hydrologic frequency analysis of form

$$Q = \overline{X} + KS \tag{3}$$

where $\overline{X}$ = mean of a random hydrologic series of variate Q

S = standard deviation

K = frequency factor which depends on recurrence interval, T, and type of probability distribution

The relationship between frequency factor, K, and recurrence interval, T, or probability of occurrence, P, for a given probability distribution can be shown either by curves or tables. The general procedure for data analysis is to determine $\overline{X}$ and S for the annual flood series and to calculate the skew coefficient if necessary for determining K. Then equation (3) is used to calculate the annual flood of magnitude Q for a selected recurrence interval or return period, T. Tables and curves for various distributions and worked examples are given in Chow (1964), McGuinness and Brakensiek (1964), and WRC (1976).

The log-Pearson Type III distribution, which is of interest herein, fits an equation similar to (3), but with the annual peak flood values transformed to the logarithm to the base 10 or

$$\log Q = \overline{X} + KS \tag{4}$$

where $\overline{X}$ = mean logarithm of hydrologic series

S = standard deviation of logarithms

K = frequency factor which depends on recurrence interval and skew coefficient

The technique of using equation (4) is described in detail in WRC (1976), so only brief discussion and a simple example are given herein.

In general, the log-Pearson Type III distribution is applied to annual flood peak discharges on unregulated streams with at least 10 years of record and ideally at least 25 years of record. There should be no probability of unusual events such as dam failures or overflow from an adjacent basin. Data come mainly from regular gaging stations or crest-height stations, but this information can be supplemented by historic data, comparison with similar watersheds, and flood estimates from precipitation. The annual flood values are assumed to come from a single population with no natural trends or effects from watershed changes. A mixed population might consist of a combination of events due to large summer rainstorms, fall hurricanes, winter rain or snow, and spring snowmelt. All of these restrictions and assumptions may, in fact, be difficult to fulfill in a State such as New Hampshire. Therefore, care must be taken in making a flood analysis.

One problem that warrants discussion is the matter of the skew coefficient which is required for determination of the frequency factor in equation (4). Because the skew coefficient requires calculation of the third power of deviations, it can be subject to large variations due to low or high outliers (values that depart considerably from the general trend) and to shortness of record (100 years are required to ease this problem). Therefore, the skew coefficient calculated from the raw data, or so-called station skew, is suspect for shorter records. Regional relationships can be developed by a study of all records in the region, with the results shown as skew isolines on a map (WRC, 1976). Then either a weighted skew coefficient or the regional skew can be used to improve matters (WRC, 1976). The WRC report also gives methods for handling features such as outliers, years of zero flood (no flow), confidence intervals, and expected probability adjustment, but these will not be considered further herein.

Annual flood discharges for a 40-year period from the Oyster River near Durham are presented in Table 1. The magnitude of each flood is given, and return period and probability are calculated according to equations (2) and (1), respectively. The data are plotted on Figure 7. The log-Pearson Type III calculations are made according to WRC (1976), and they are summarized in Table 2. The results are shown as a solid line on Figure 7. A next step in utilizing this kind of information is to convert flood discharge to elevation so that a flood zone of any desired probability can be delineated on a map. A discussion of this process is beyond the scope of this report; however, guidelines and references may be found in the reports listed in Chapter I.

Table 1. Oyster River Flood Peaks for 1935-1974 in cfs

Year	Magnitude	T, years	P	cfs	Year	Magnitude	T, years	P	cfs
1935	18	2.28	.439	345	1955	14	2.93	.341	374
1936	3	13.70	.073	548	1956	17	2.41	.415	351
1937	15	2.73	.366	369	1957	40	1.02	.976	91
1938	8	5.12	.195	422	1958	9	4.56	.220	400
1939	35	1.17	.854	162	1959	24	1.71	.585	260
1940	19	2.16	.463	334	1960	7	5.85	.171	427
1941	25	1.64	.610	250	1961	20	1.37	.732	213
1942	13	3.15	.317	380	1962	11	3.73	.268	386
1943	16	2.56	.390	355	1963	5	8.20	.122	450
1944	34	1.20	.829	165	1964	31	1.32	.756	213
1945	28	1.46	.683	217	1965	38	1.08	.927	110
1946	29	1.41	.707	215	1966	39	1.05	.951	106
1947	33	1.24	.805	168	1967	20	2.05	.488	309
1948	21	1.95	.512	300	1968	6	6.83	.146	440
1949	36	1.14	.878	144	1969	26	1.58	.634	240
1950	12	3.42	.293	384	1970	22	1.86	.536	280
1951	23	1.78	.561	261	1971	37	1.11	.902	140
1952	10	4.10	.244	389	1972	27	1.52	.658	233
1953	4	11.20	.098	498	1973	2	20.50	.049	610
1954	1	41.00	.024	862	1974	32	1.28	.780	169

Table 2. Calculations for Log-Pearson Type III Fit to Oyster River Annual Flood Peaks

Mean of Logarithms, $\overline{X}$ = 2.44681 (Period of record = 40 years)

Standard Deviation of logs, S = 0.21738

Station Skew Coefficient, G = -0.3

Generalized Skew Coefficient = +0.58

Weighted Skew Coefficient = $\left(\frac{40-25}{75}\right)$ (-0.3) + $\left(1 - \frac{40-25}{75}\right)$ (0.58) = 0.4

$$\log Q = 2.44681 + 0.21738\ (K_{.4,\ p})$$

P	T, years	$K_{.4,\ p}$	log Q	Q, cfs
.99	1.01	-2.02933	2.00567	101
.90	1.11	-1.23114	2.17918	151
.50	2.00	-0.06651	2.43235	271
.10	10.00	1.31671	2.73304	541
.05	20.00	1.75048	2.82733	672
.02	50.00	2.26133	2.93838	862
.01	100.00	2.61539	3.01534	1040
.005	200.00	2.94900	3.08786	1220
.002	500.00	3.36566	3.17844	1510

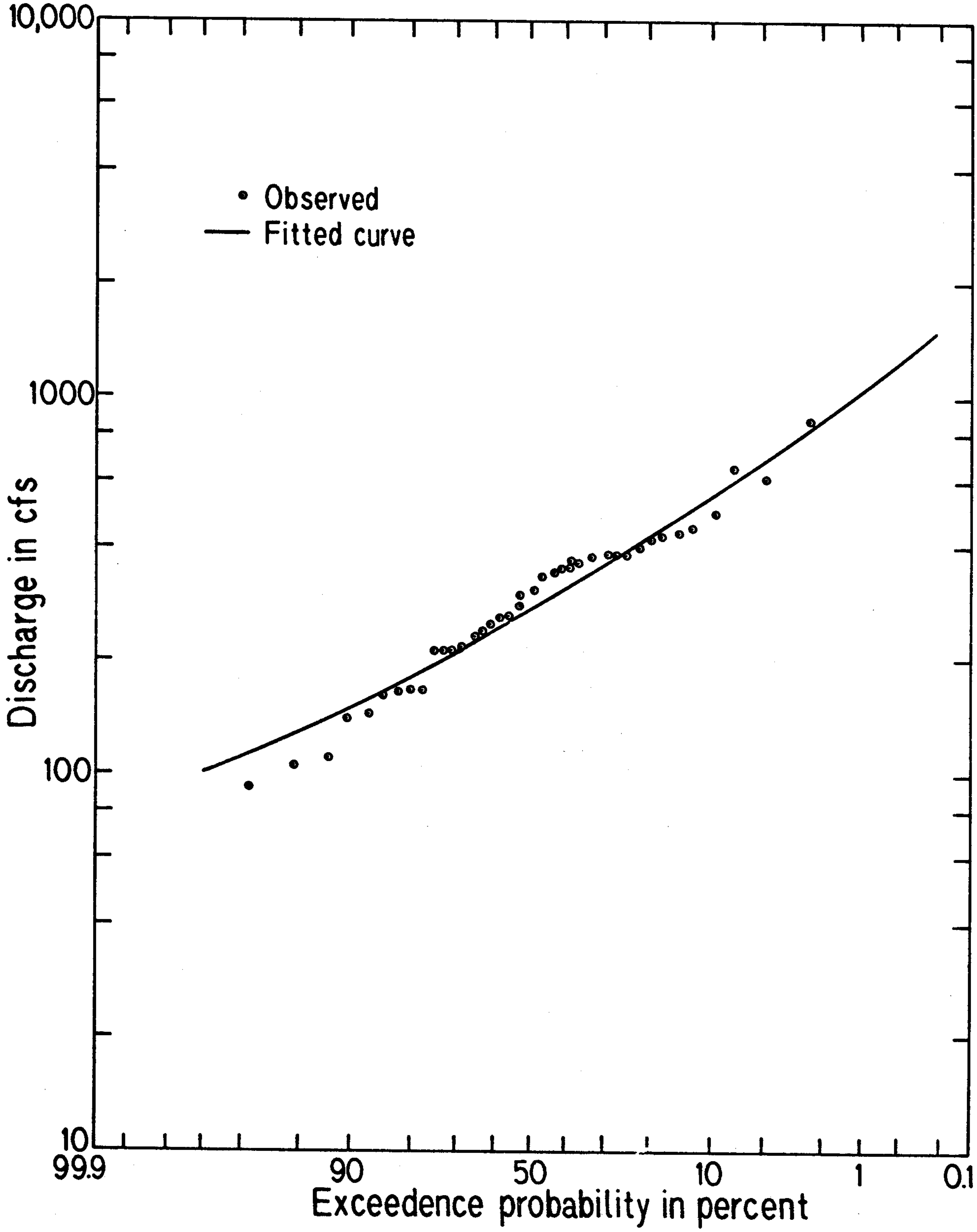

Figure 7. Oyster River Near Durham, N.H.: 1935-1974.
Annual Flood Peaks and Log-Pearson Type III Curve.

Chapter III

Section I

ESTIMATION OF FLOW-DURATION CURVES FOR UNREGULATED STREAMS IN NEW HAMPSHIRE

S. Lawrence Dingman[1]

INTRODUCTION

A flow-duration curve is a graph that shows the frequency, or probability, that a given mean daily streamflow (usually expressed in cubic feet per second) will be equaled or exceeded at a specified point on a stream. It is thus a concise "picture" of the variability of streamflow at that point. For example, Figure 8 shows the flow-duration curve for the Oyster River at the U.S. Geological Survey stream-gaging station near Durham, New Hampshire. Following the dashed lines, this graph shows that 90% of the time, the flow there is equal to or greater than 1.15 ft^3/s, 50% of the time it is equal to or greater than 9.5 ft^3/s, and 10% of the time it is equal to or greater than 49 ft^3/s.

A flow-duration curve is one of the most useful types of information for use in water resources planning. Knowing the frequency with which various flow rates occur is invaluable data for assessing water available for municipal or industrial water supplies, for dilution of waste-treatment plant effluents and cooling water, for generation of hydroelectric power, for fish and other wildlife, and for navigation. As a simple example of the use of these curves, suppose it was desired to use the Oyster River as a municipal water supply. Assume further that a general policy was established that the supply had to be adequate 95% of the time. From the graph, the flow available

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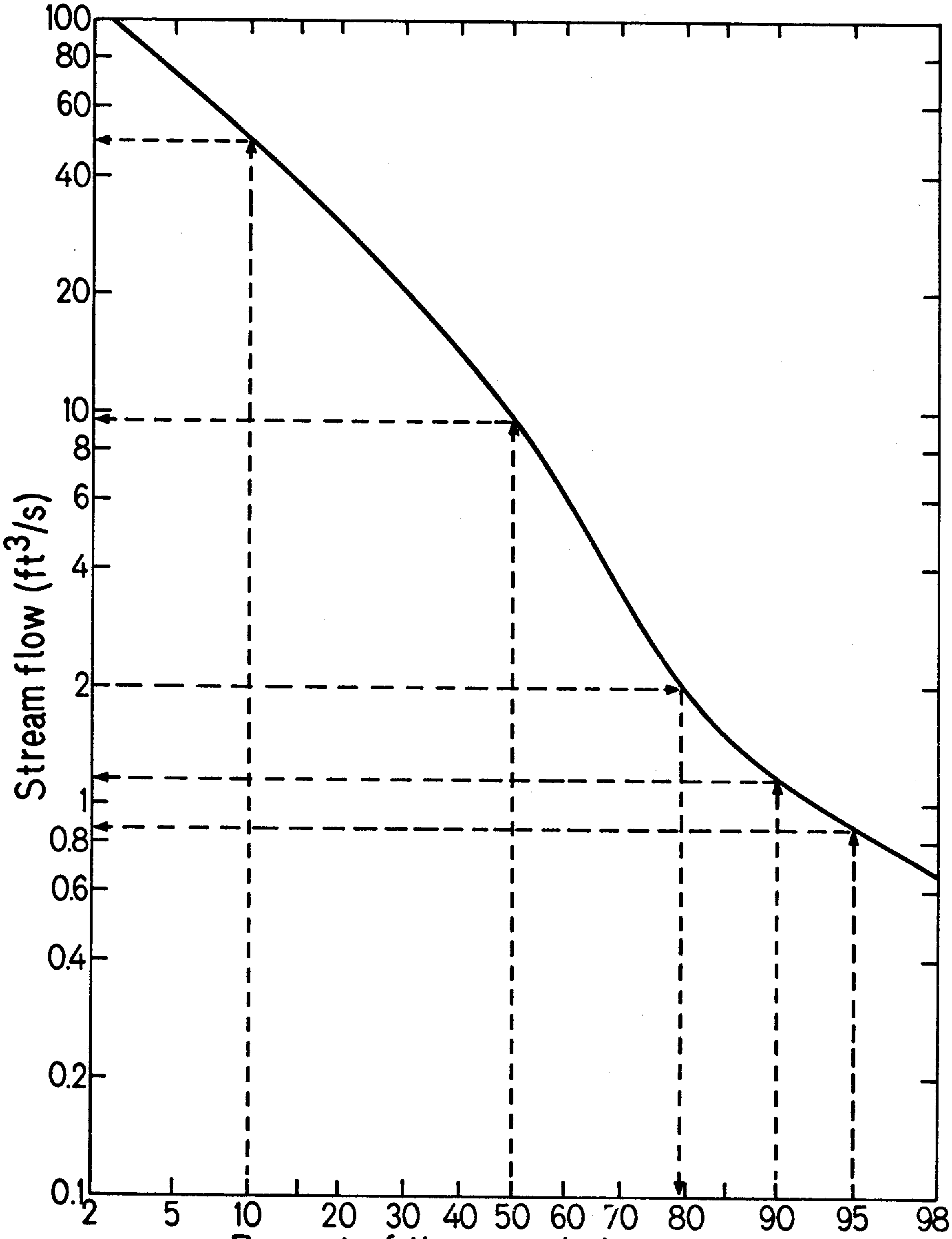

Figure 8. Flow-Duration Curve for the Oyster River Near Durham, New Hampshire. Dashed lines refer to examples of use given in text.

95% of the time is 0.86 ft^3/s, which is the same as 385 gal/min. If it is assumed that each person uses 100 gal/day, the flow at this point on the Oyster River is adequate for a population of 5544. (In this simple example, it is assumed that it would be acceptable to use all the flow of the river 5% of the time.) Or, suppose that the discharge from a waste-treatment plant at this point required a flow of 2 ft^3/s for dilution to acceptable water-quality levels. The graph shows that such a flow would be available only about 79% of the time, and a decision would have to be made as to whether lower water quality could be tolerated 21% of the time, or whether further treatment or a different plant location was necessary.

Another potentially significant use of flow-duration curves is for regulatory purposes. The U.S. Army Corps of Engineers has proposed that all streams in which streamflow is less than 5 ft^3/s for more than six months a year be exempt from permits under the dredge-and-fill section of the Water Pollution Control Act Amendments of 1972 (Sec. 404 of P.L. 92-500). A flow-duration curve would immediately reveal exempt and non-exempt stream reaches. For example, Figure 8 shows that 5 ft^3/s is exceeded about 65% of the time at the Oyster River gage, so that the proposed rule would require permits for dredge-and-fill activity on the river below the gage as well as upstream to the point where 5 ft^3/s flow is exceeded 50% of the time. A few trial-and-error attempts using the method described here would suffice to identify exempt and non-exempt reaches of any given stream.

At points where streamflow is continuously monitored by the U.S. Geological Survey, flow-duration curves are developed by analyzing the daily streamflow records, as described in Chapter II. Generally, at least 10 years of continuous daily streamflow data are required to develop representative curves. The Geological Survey periodically produces flow-duration information for the approximately 50 gaging stations it maintains in New Hampshire, and this is available in their files. However, information on flow variability is often needed for water-resources planning at points where no streamflow measurements have been made. Thus the objective of this chapter is to describe a method by which one can make useful estimates of flow-duration curves at points where no streamflow data have been collected, using information which can be readily obtained for any point in the State. Because the operation of reservoirs or the existence of lakes where the residence time or storage ratio* exceeds one day has a complicating effect on flow-duration curves, the method described here is suitable only for unregulated streams. Further research will be needed to account for the effects of regulation.

The method developed here is strictly applicable only to New Hampshire, but there is reason to believe that this general approach to flow-duration curve synthesis is valid at least throughout northern New England (Dingman, 1978), and very likely in other regions where significant elevational gradients of climatic factors exist.

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*Residence time or storage ratio is calculated by dividing the reservoir or lake volume by the flow rate. For example, a 50-acre pond that has an average depth of 10 feet has a volume of 21,780,000 ft^3. At a flow rate of 5 ft^3/s, the residence time is 4,356,000 seconds or 50.4 days.

METHOD

The procedure for estimating flow-duration curves is described in step-by-step manner below. Section II completely describes the procedure used in developing the method, gives procedures for calculating the reliability of estimates, presents results of a test of the method, and discusses the hydroclimatological processes that underlie the equations presented below. Section III contains a computer program written in BASIC language that calculates flow-duration-curve parameters and their confidence intervals using the method described below.

1. The only material needed to estimate flow-duration curves beyond the information provided here is a topographic map covering the drainage basin of the stream being studied and a means for measuring the area of this basin. After the point on the stream for which flow-duration information is needed is identified, the drainage basin divides are traced out on the basis of the contours. For drainage basins over about 20 mi^2, the U.S. Geological Survey 1:250,000-scale maps are satisfactory, while for smaller basins the standard 15-minute quadrangles at a scale of 1:62,500 should be used. To measure area, either a planimeter or a grid-point-counting method can be used. In the equations presented below, the drainage-basin area has the symbol A_D, and must be expressed in mi^2.

2. Once the boundaries of the drainage basin are traced out and the area measured, the next step is to identify the elevation of the highest and lowest points in the basin. The highest point will generally, but not always, be located somewhere on the drainage divide, and the lowest point is always the point on the stream for which information is desired.

In the equations presented below the highest and lowest elevations have the symbols Y_{max} and Y_{min}, respectively, and must be expressed in feet.

3. Estimate the average elevation of the drainage basin $\hat{Y}$ from equation 5:

$$\hat{Y} = 0.324\ (Y_{max} - Y_{min}) + Y_{min} \qquad (5)$$

As noted in Section II, better estimates can be developed using the actual measured mean basin elevation rather than the estimated value from equation 5.

4. From Figure 9, determine which hydrologic region the drainage basin is in.

5. Depending on the hydrologic region, use one of the following equations to estimate the mean flow, $\overline{Q}$, in ft^3/s:

$$\text{Region I:}\quad \overline{Q} = (1.30 + 0.000515\ \hat{Y})\ A_D \qquad (6\text{-I})$$

$$\text{Region II:}\quad \overline{Q} = (1.01 + 0.000398\ \hat{Y})\ A_D \qquad (6\text{-II})$$

$$\text{Region III:}\quad \overline{Q} = (1.19 + 0.000383\ \hat{Y})\ A_D \qquad (6\text{-III})$$

6. Use equation 7 to estimate the flow that is equaled or exceeded 30% of the time, Q_{30}, in ft^3/s:

$$Q_{30} = 0.880\overline{Q} \qquad (7)$$

7. Use equation 8 to estimate the flow that is equaled or exceeded 5% of the time, Q_{05}, in ft^3/s:

$$Q_{05} = 3.90\overline{Q} \qquad (8)$$

8. Use equation 9 to estimate the flow that is equaled or exceeded 2% of the time, Q_{02}, in ft^3/s:

$$Q_{02} = 6.00\overline{Q} \qquad (9)$$

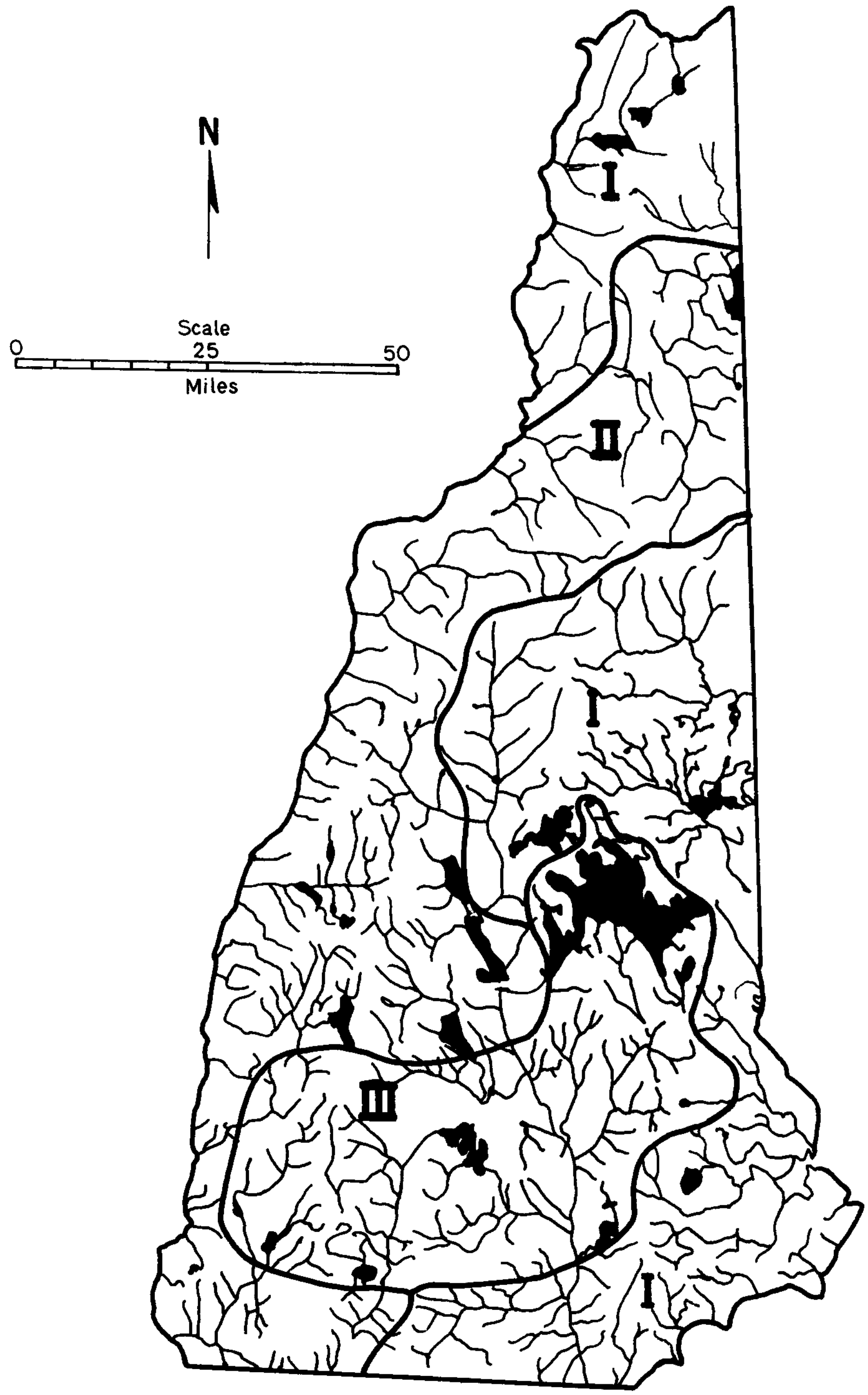

Figure 9. Hydrologic Regions for Estimation of Average Streamflow (Equation 6).

9. Use equation 10 to estimate the flow in ft^3/s that is equaled or exceeded 95% of the time, Q_{95}:

$$Q_{95} = [0.0796 - (0.107)(10^{-3})\hat{Y} + (0.901)(10^{-7})\hat{Y}^2]\ A_D \tag{10}$$

10. On three-cycle log-probability paper, plot $\overline{Q}$ at the 27% exceedance frequency, and Q_{02}, Q_{05}, Q_{30}, and Q_{95} at their respective exceedance frequencies. Sketch a smooth line through the points from Q_{02} to Q_{30}, continuing the trend to about the 50% exceedance line. Then begin a smooth curve so that the line intersects Q_{95}; continue the line to Q_{98}.

11. Section II presents methods whereby the confidence intervals associated with the estimates of Q_{02}, Q_{05}, $\overline{Q}$, Q_{30}, and Q_{95} can be computed. As noted there, confidence intervals for low-flow estimates are likely to be particularly important, as design decisions with economic implications are commonly made on the basis of information on low flows. In these cases, the use of a measured value of mean basin elevation is recommended rather than the estimate using equation 5. However, if information about low flows is of especially critical importance at a site, it is recommended that field measurements of discharge be made during the low-flow season. The measurements can then be compared (as a ratio to drainage area or to average flow as estimated from equations 6-I, 6-II, 6-III) to simultaneous flows at a nearly gaged stream to get a firmer picture of the actual streamflow variability. In order for such comparisons to be valid, however, these measurements must be made at least several days after significant rainfall on the two watersheds, and any anomalous behavior due to storage effects (lakes, reservoirs) avoided or accounted for.

Chapter III

Section II

SYNTHESIS OF FLOW-DURATION CURVES FOR UNREGULATED STREAMS IN NEW HAMPSHIRE

S. Lawrence Dingman

APPROACH

Ongoing investigations of the hydrology of northern New England indicated several features of potential usefulness for synthesizing flow-duration curves: 1) mean streamflow per unit area from a drainage basin is highly correlated with the mean elevation of the basin (Dingman, 1978); 2) the upper portions of flow-duration curves for gaging stations in New Hampshire show certain consistencies of shape (Ives, 1977); and 3) the low ends of flow-duration curves for gaging stations do not appear to be related to basin geology (Ives, 1977). This latter fact is in surprising contrast to general belief (e.g. Searcy, 1959) and to the results of several earlier studies (Thomas, 1966; Ackroyd et al., 1967; Ayers and Ding, 1967).

Consideration of the above features suggested the overall approach applied herein: 1) develop a method for estimating mean basin elevation; 2) establish relations between mean flow and mean basin elevation; 3) quantify the regular features of the upper portions of flow-duration curves; and 4) relate low ends of flow-duration curves to readily determinable basin parameters. As shown below, mean basin elevation also turned out to be the best predictor of low flows.

ESTIMATION OF MEAN BASIN ELEVATION

Measurement of mean basin elevation requires construction of an area-elevation curve for the basin. This is a tedious and time-consuming task, especially for large areas. An expedient means for estimating mean elevation was suggested by Langbein et al. (1947), who reported on an extensive analysis of the topographic features of gaged basins in the eastern United States. Those authors prepared a large number of area-elevation curves, and found that "the variations are wide, but in general the mean altitude of a basin is located at 0.34 of the range between minimum and maximum..." (Langbein et al., 1947, p. 140-141).

In the present study, actual mean elevations ($\overline{Y}$) were determined from area-elevation curves prepared for 10 smaller (2.94 to 12.1 mi^2) gaged basins, and were combined with the values for the 19 New Hampshire basins studied by Langbein et al. (1947) to make a sample of 29 for the State. Figure 10 shows the location of the gaging stations, and Table 3 lists the elevation data for each. For this sample, the average location of the mean basin elevation was 0.324 of the distance between the minimum, Y_{min}, and maximum elevation, Y_{max}, with a range of from 0.215 to 0.479. Assuming a normal distribution for the State, 95% of the basins will have a value between 0.180 and 0.468. Further analysis revealed no identifiable relation between this value and drainage area, gage elevation, or geographic location.

Thus, for the purposes of subsequent analyses, the estimated mean elevation, $\hat{Y}$, was determined for all gaged basins using the formula

$$\hat{Y} = Y_{min} + 0.324\ (Y_{max} - Y_{min}) \tag{5}$$

These values are listed in Table 3.

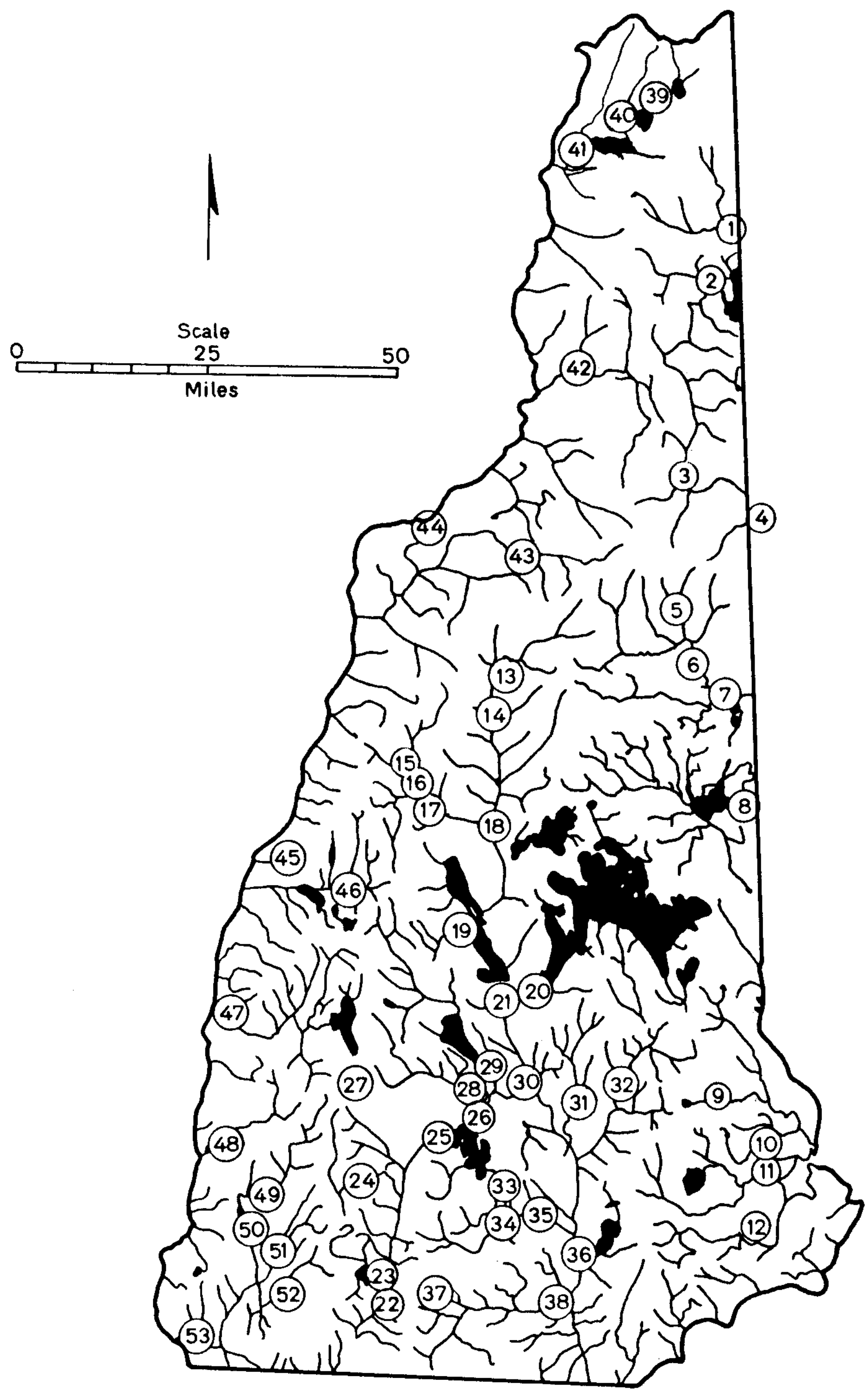

Figure 10. Location of Gaging Stations for Which Data on Mean Elevation, Mean Streamflow, and Streamflow Variability were Analyzed. See Tables 3, 5, 6, and 7.

Table 3. Tabulation of Data Used in Analysis of Mean Elevation and Mean Flow

Sta. No.[1]	Stream	at/near	Drainage Area (mi^2)	Elevations (ft) Y_{min}	Y_{max}	Measured Mean, $\overline{Y}$	Est. Mean, $\hat{Y}$	Average Streamflow $\overline{Q}_{in}$ (in/yr)
1	Diamond R.	Wentworth Loc.	153	1275	3607		2030	30.80
2	Androscoggin R.	Erroll	1045	1227	4180		2180	24.67
3	Androscoggin R.	Gorham	1363	833	4180		1920	24.56
4	Wild R.	Gilead, Me.	69.5	683	4832		2030	34.39
5	Ellis R.	Jackson	10.9	1500	6288	3340	3050	43.11
6	Lucy Bk.	N. Conway	4.68	710	3201	1540	1520	31.34
7	Saco R.	Conway	386	418	6288		2320	32.47
8	Ossipee R.	Effingham Falls	330	390	3993		1560	28.39
9	Mohawk Bk.	C. Strafford	8.87	285	1425	590	650	18.53
10	Oyster R.	Durham	12.1	70	365	200	170	21.66
11	Lamprey R.	Newmarket	183	40	1413		480	20.63
12	Dudley Bk.	Exeter	4.97	90	265	140[2]	150	19.26
13	E. Br. Pemigewasset R.	Lincoln	104	1020	5249	2800[2]	2390	
14	Pemigewasset R.	Woodstock	193	615	5249	2490[2]	2120	35.89
15	Baker R.	Wentworth	58.8	580	4810	1740[2]	1950	
16	Stevens Bk.	Wentworth	2.94	595	3390	1630[2]	1500	21.66
17	Baker R.	Rumney	143	495	4810	1580[2]	1890	24.03
18	Pemigewasset R.	Plymouth	622	457	5249	1850[2]	2010	29.43
19	Smith R.	Bristol	85.8	450	2920	1260[2]	1250	22.48
20	Winnipesaukee R.	Tilton	471	442	2982		1260	19.98
21	Merrimack R.	Franklin Jct.	1507	250	5249		1870	24.83
22	Contoocook R.	Peterborough	68.1	740	3165		1530	23.13
23	Nubansit Bk.	Peterborough	46.9	790	2233		1260	23.86
24	N. Br. Contoocook R.	Antrim	54.8	880	2496	1540[2]	1400	24.63
25	Contoocook R.	Henniker	368	475	3165		1350	23.14
26	Contoocook R.	W. Hopkinton	427	355	3165		1270	21.56
27	W. Br. Warner R.	Bradford	5.75	950	2500	1500[2]	1450	25.74
28	Warner R.	Davisville	146	380	2743	970[2]	1150	21.77
29	Blackwater R.	Webster	129	430	2937	1100[2]	1240	22.21
30	Contoocook	Penacook	766	273	3165	1040[2]	1210	22.23

Table 3. (Cont.)

Sta.[1] No.	Stream	at/near	Drainage Area (mi^2)	Elevations (ft) Y_{min}	Y_{max}	Measured Mean, $\overline{Y}$	Est. Mean, $\hat{Y}$	Average Streamflow $\overline{Q}_{in}$ (in/yr)
31	Soucook	Concord	76.8	290	1506		680	19.45
32	Suncook	N. Chichester	157	340	2378	820[2]	1000	20.50
33	Piscataquog	E. Weare	63.1	320	1522		710	19.63
34	S. Br. Piscataquog	Goffstown	104	310	2055		889	21.41
35	Piscataquog	Goffstown	202	185	2055		790	20.44
36	Merrimack	Goffs Falls	3092	109	5249		1770	22.94
37	Stony Bk. Trib.	Temple	3.60	920	2300	1390	1370	25.05
38	Souhegan	Merrimack	171	161	2279	810[2]	850	22.47
39	Big Bk.	Pittsburg	6.36	1680	3168	2150	2060	32.24
40	Connecticut R.	First Lake	83.0	1560	3643		2230	32.07
41	Connecticut R.	Pittsburg	254	1150	3643		1960	30.26
42	Upper Ammonoosuc R.	Groveton	232	920	4165		1970	27.98
43	Ammonoosuc R.	Bethleham Jct.	87.6	1181	6288	2510[2]	2840	31.93
44	Ammonoosuc R.	Bath	395	454	6288	1710[2]	2340	22.45
45	Mink Bk.	Etna	4.60	1000	2290	1450	1420	18.95
46	Mascoma R.	West Canaan	80.5	835	3240	1400[2]	1610	19.74
47	Sugar R.	W. Claremont	269	359	2743	1250[2]	1130	19.99
48	Cold R.	Drewsville	82.7	375	2182		960	18.88
49	Ashuelot R.	Gilsum	71.1	773	2332	1520[2]	1280	23.68
50	Ashuelot R.	Keene	101	480	2332		1080	22.86
51	Otter Bk.	Keene	47.2	659	2153		1140	21.49
52	S. Br. Ashuelot	Webb	36.0	667	3165	1280[2]	1480	22.18
53	Ashuelot	Hinsdale	420	201	3165	1200[2]	1160	21.28

[1] Fig. 10 shows locations

[2] Mean elevation calculated by Langbein et al. (1947)

MEAN STREAMFLOW AND MEAN ELEVATION

An initial linear regression analysis was carried out between $\hat{Y}$ and the long-term average flow, $\overline{Q}_{in}$, for 50 of the New Hampshire stations. $\overline{Q}_{in}$ represents the average flow for the period of record through water-year 1974 in 48 cases, and through 1970 in two cases (North Branch Contoocook River and Suncook River). The Wild River at Gilead, Maine, and two stations at which gaging was discontinued in the early 1950s (East Branch Pemigewasset River and Baker River at Wentworth), were not included in this analysis. Table 4 shows the resulting regression equation (equation 11); the correlation coefficient is significant at the 0.001 level.

However, further analysis was attempted to see if the standard error of estimate could be reduced. This was accomplished by plotting residuals of equation 11 ($\overline{Q}_{in}$ observed minus $\hat{Q}_{in}$ estimated) on a river-basin map of the State. These residuals showed a definite geographic pattern such that three hydrologic regions could be identified (Figure 9); in Region I, the residuals generally exceeded +1.0 in./yr; in Region II, they exceeded (in the negative direction) -1.0 in./yr; and in Region III, they were generally between -1.0 in./yr and +1.0 in./yr. The regional boundaries generally follow divides of major river basins. Thus it was hypothesized that the stations in each region represented separate populations, and separate regression equations were developed for each, with the results shown in Table 4 (equations 12-I - 12-III). Application of t-tests showed that equations 12-I - 12-III were all significantly different from each other at the 0.05 level, confirming the hypothesis. Figure 11 shows the data points and regression lines for equations 12-I - 12-III.

Table 4. Regression Equations Relating Long-Term Mean Flow, Q_{in} (in/yr), and Estimated Mean Basin Elevation, $\hat{Y}$ (ft)

Eq.	Sample	n	Equation	r	t	std. error (in/yr)
(11)	all stations	50	$\hat{Q}_{in} = 15.0 + 0.00647\ \hat{Y}$	.778	8.59	3.23
(12-I)	Region I	16	$\hat{Q}_{in} = 17.7 + 0.00697\ \hat{Y}$	.931	9.57	2.32
(12-II)	Region II	16	$\hat{Q}_{in} = 13.7 + 0.00536\ \hat{Y}$	.837	5.73	1.81
(12-III)	Region III	18	$\hat{Q}_{in} = 16.2 + 0.00518\ \hat{Y}$	.824	5.83	1.10

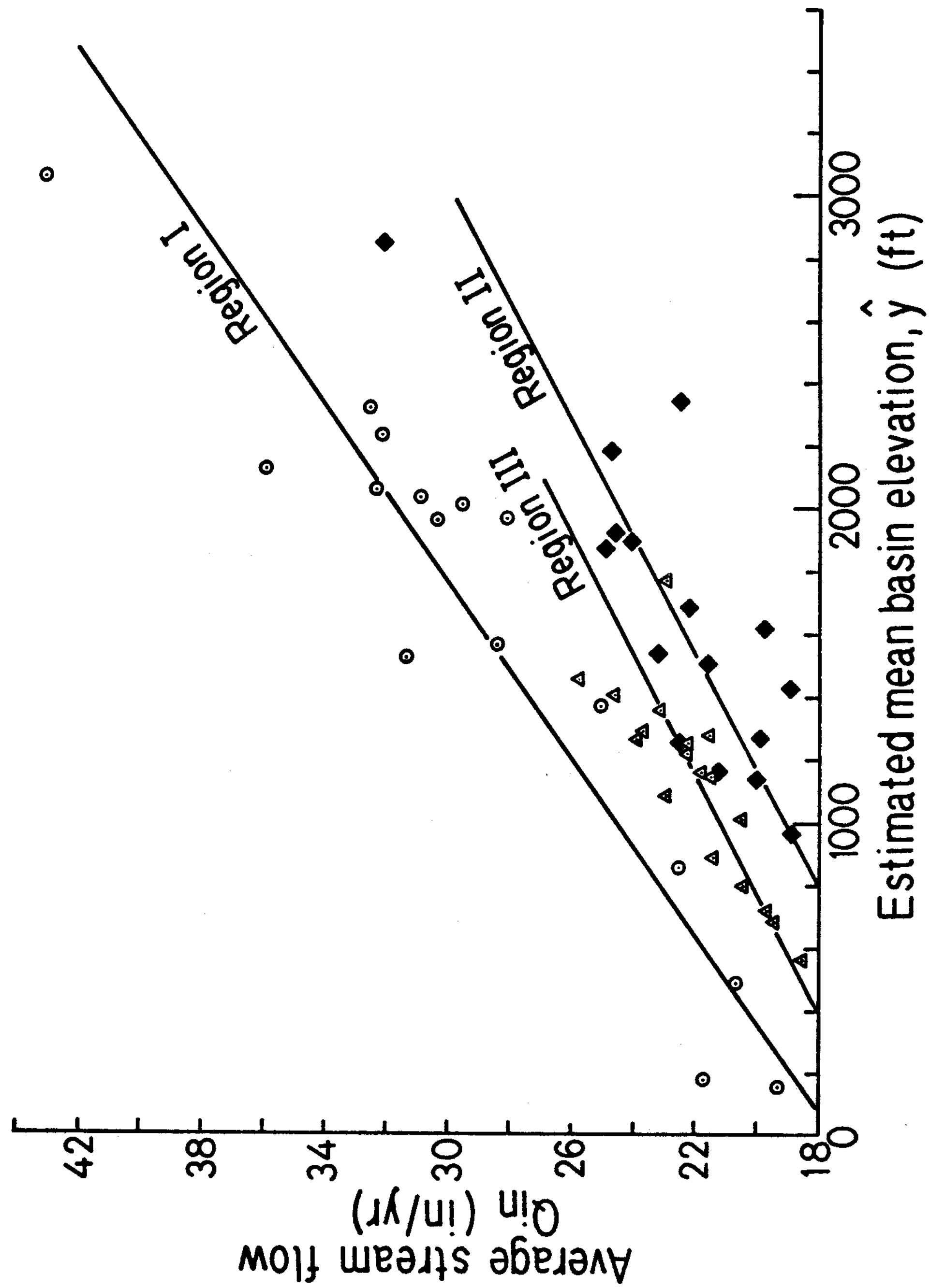

Figure 11. Relations Between Average Streamflow and Estimated Mean Basin Elevation for the Three Hydrologic Regions of New Hampshire. Regression equations are given in Table 4.

CHARACTERIZATION OF UPPER PORTIONS OF FLOW-DURATION CURVES

Of the sample of 53 streams in Table 3, 24 are unaffected by regulation and have records exceeding 10 years, and hence have flow-duration curves suitable for analysis herein. In order to provide a valid test of the method being developed, one stream from each of three drainage-area classes (< 20 mi^2, 20 to 120 mi^2, and > 120 m^2) was randomly selected, and its flow-duration curve was eliminated from subsequent analysis. The streams selected were Big Brook, Smith River, and Pemigewasset River at Woodstock.

Flow-duration curves for the remaining 21 stations were plotted and their upper portions characterized by the eight parameters shown in Table 5. The average values and standard deviations were calculated for each, and a consistency ranking computed on the basis of coefficients of variation. The four least variable parameters were then selected to characterize the upper portions of the curves: 1) the exceedance frequency of the mean flow, $f_{\overline{Q}}$; 2) the ratio of the flow exceeded 5% of the time to mean flow, $Q_{05}/\overline{Q}$; 3) the ratio of the flow exceeded 30% of the time to mean flow, $Q_{30}/\overline{Q}$, and; 4) the ratio of the flow exceeded 2% of the time to mean flow, $Q_{02}/\overline{Q}$. It was assumed that the average values of these parameters calculated for this sample would apply for estimated flow-duration curves. The 95% confidence intervals for the parameters (assuming they are normally distributed) are also shown in Table 5.

CHARACTERIZATION OF LOW ENDS OF FLOW-DURATION CURVES

As noted earlier, it is generally believed that the low-flow ends of flow-duration curves are largely controlled by basin geology, through

Table 5. Data Characterizing High Ends of Flow-Duration Curves

Stream	$f_{\overline{Q}}$ (%)	Q_{02}/A_D $\frac{ft^3}{s\ mi^2}$	$Q_{02}/\overline{Q}$	Q_{05}/A_D $\frac{ft^3}{s\ mi^2}$	$Q_{05}/\overline{Q}$	$Q_{30}/\overline{Q}$	Q_{02}/Q_{30}	Q_{05}/Q_{30}
Diamond R.	24	14.9	6.4	9.6	4.1	.85	7.5	4.8
Wild R.	24	17.3	6.9	9.6	3.9	.73	9.4	5.3
Ellis R.	26	17.6	5.6	11.0	3.5	.86	6.5	4.1
Lucy Bk.	24	12.8	5.6	8.7	3.8	.76	7.4	5.0
Saco R.	26	13.5	5.6	8.7	3.6	.85	6.6	4.2
Mohawk Bk.	29	8.4	6.2	5.0	3.7	.97	6.4	3.8
Oyster R.	29	8.3	5.5	7.4	4.9	.97	5.7	5.0
Dudley Bk.	27	9.5	6.8	5.3	3.8	.84	8.1	4.5
Stevens Bk.	24	11.4	7.2	7.0	4.4	.71	10.1	6.2
Baker R. (Rumney)	25	10.1	5.7	6.9	3.9	.81	7.1	4.8
Pemigewasset R. (Plymouth)	27	11.9	5.5	7.7	3.6	.89	6.2	4.0
W. Br. Warner R.	25	13.9	7.1	8.0	4.1	.81	8.8	5.1
Warner R.	30	8.9	5.6	6.0	3.7	.99	5.6	3.7
Soucook R.	29	7.9	5.7	5.2	3.7	.99	5.8	3.7
S. Br. Piscataquog R.	30	8.7	5.5	5.9	3.7	.99	5.5	3.7
Stony Bk. trib.	29	11.7	6.2	7.2	3.8	.97	6.4	3.9
Ammonoosuc R. (B. Jct.)	26	12.6	5.4	8.2	3.5	.84	6.4	4.2
Mink Bk.	29	8.7	6.0	5.7	3.9	.95	6.3	4.1
Mascoma R.	27	8.0	5.7	5.2	3.7	.88	6.5	4.2
Cold R.	27	8.3	6.0	5.4	3.9	.87	6.9	4.5
S. Br. Ashuelot R.	28	9.6	5.9	6.1	3.7	.95	6.2	3.9
Average	27	11.5	6.0	7.3	3.9	.88	7.0	4.4
Standard Deviation	1.96	3.07	0.56	1.73	0.30	0.082	1.18	0.61
Coefficient of Variation	0.073	0.27	0.093	0.24	0.077	0.093	0.17	0.14
Consistency Ranking	1	8	4	7	2	3	6	5
Upper 95% conf. limit*	30.8	17.5	7.1	10.7	4.5	1.04	9.3	5.6
Lower 95% conf. limit*	23.2	5.5	4.9	3.9	3.3	0.72	4.7	3.2

*Assuming normal distributions

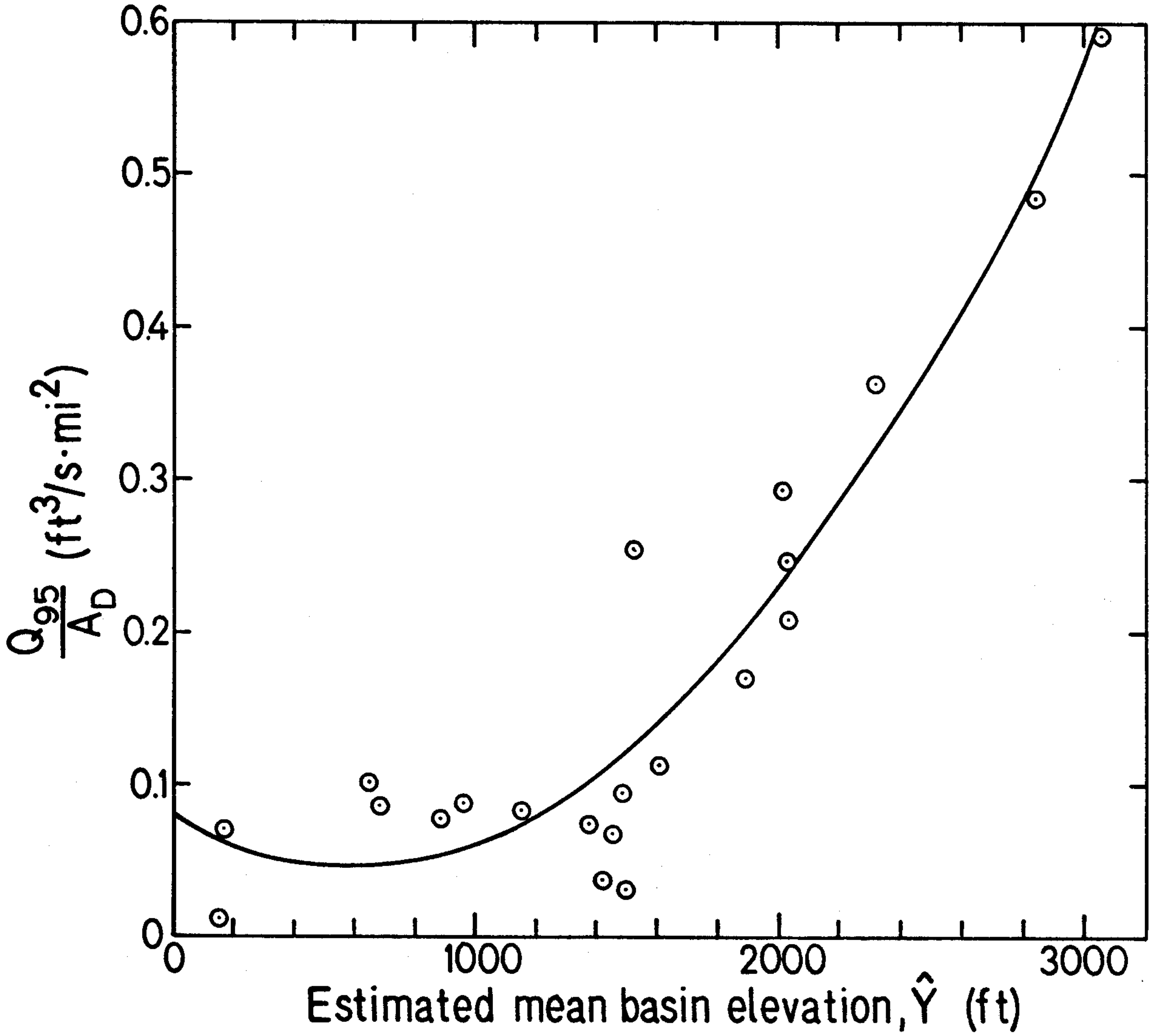

Figure 12. Relation Between Streamflow Exceeded 95% of the Time and Estimated Mean Basin Elevation for New Hampshire. Curve is Polynomial Regression Equation 13.

its effect on infiltration and transmissivity (Searcy, 1959). Several studies have demonstrated such a relationship (Ackroyd et al., 1967; Ayers and Ding, 1967); Thomas (1966) found a particularly striking relation between glacial geology and flow-duration curves for streams in Connecticut. However, in a study preliminary to the present work, Ives (1977) could find no indication that geology or soils were significantly related to low flows in New Hampshire. Similarly, although other studies had claimed relationships between low flows and geomorphologic parameters, especially drainage density (e.g., Carlston, 1963; Orsborn, 1976), extensive trials using various combinations of drainage density, slope, and relief for New Hampshire streams proved fruitless. However, the ratio of the flow exceeded 95% of the time to drainage area, Q_{95}/A_D ($ft^3/s\ mi^2$), was significantly correlated to estimated mean basin elevation. The best estimating equation was found to be:

$$Q_{95}/A_D = 0.0796 - (0.107)(10^{-3})\hat{Y} + (0.901)(10^{-7})\hat{Y}^2 \qquad (13)$$

$$n = 21,\ r = 0.945,\ t = 12.3,\ \text{std. error} = 0.0532\ ft^3/s\ mi^2$$

which is plotted in Figure 12.

The data plotted in Figure 12 suggest two populations: 1) basins with $\hat{Y} < 1500$ ft., where Q_{95}/A_D does not depend on $\hat{Y}$, and; 2) basins with $\hat{Y} > 1500$, which show a strong relationship between Q_{95}/A_D and $\hat{Y}$. However, because of the relatively small sample size, it was found that use of the second-degree polynomial expression of equation 13 provided the smallest confidence intervals for predicting Q_{95}/A_D.

Estimation of Reliability

A potential user of a prediction method requires information about the accuracy of the method, i.e., the likelihood that an estimate is close to the actual value. This accuracy can be expressed as confidence intervals (or bands) about the estimated values; these can be calculated for the method presented herein on the basis statistical parameters of regression equations and sample standard deviations (Crow _et al._, 1960). The following steps show how the 95% confidence bands for flow-duration curves estimated for New Hampshire streams can be calculated. Actual flow-duration curves will lie within these bands in 95% of the future estimates made following the method described here. These confidence intervals can be significantly reduced by eliminating step 1 and using a single measured value of $\overline{Y}$ in the following procedure.

1. As noted earlier, the data indicate that we are 95% confident that the true mean elevation of a drainage basin lies between the following limits:

$$\hat{Y}_{low} = Y_{min} + 0.180\ (Y_{max} - Y_{min}) \tag{14}$$

$$\hat{Y}_{high} = Y_{min} + 0.468\ (Y_{max} - Y_{min}) \tag{15}$$

Thus the first step is to calculate $\hat{Y}_{low}$ and $\hat{Y}_{high}$ by equations 14 and 15.

2. Calculate the $\hat{Q}_{low}$ and $\hat{Q}_{high}$ from the appropriate mean-flow equation (equation 12-I - 12-III or 6-I - 6-III) using $\hat{Y}_{low}$ and $\hat{Y}_{high}$, respectively. Calculate the upper ($\hat{Q}^{+}$) and lower ($\hat{Q}^{-}$) 95% confidence limits as follows (see Crow _et al._, 1960, p. 163):

<u>For Region I:</u>

$$\hat{Q}^{-} = \hat{Q}_{low} - \left[0.367 \sqrt{1.06 + \frac{(\hat{Y}_{low} - 1616)^2}{(1.01)(10^7)}} \right] A_D \tag{16}$$

$$\hat{Q}^{+} = \hat{Q}_{high} + \left[0.367\sqrt{1.06 + \frac{(\hat{Y}_{high} - 1616)^2}{(1.01)(10^7)}} \right] A_D \tag{17}$$

For Region II:

$$\hat{Q}^{-} = \hat{Q}_{low} - \left[0.286\sqrt{1.06 + \frac{(\hat{Y}_{low} - 1646)^2}{(3.77)(10^6)}} \right] A_D \tag{18}$$

$$\hat{Q}^{+} = \hat{Q}_{high} + \left[0.286\sqrt{1.06 + \frac{(\hat{Y}_{high} - 1646)^2}{(3.77)(10^6)}} \right] A_D \tag{19}$$

For Region III:

$$\hat{Q}^{-} = \hat{Q}_{low} - \left[0.172\sqrt{1.06 + \frac{(\hat{Y}_{low} - 1128)^2}{(1.52)(10^6)}} \right] A_D \tag{20}$$

$$\hat{Q}^{+} = \hat{Q}_{high} + \left[0.172\sqrt{1.06 + \frac{(\hat{Y}_{high} - 1128)^2}{(1.52)(10^6)}} \right] A_D \tag{21}$$

Plot $\hat{Q}^{-}$ and $\hat{Q}^{+}$ below and above the estimate of $\hat{Q}$.

3. From Table 5, we are 95% confident that the mean flow occurs between the 23.2% and 30.8% exceedance frequencies. This information allows the plotting of a rectangle within which we are 95% confident the mean flow occurs.

4. Again using information from Table 5, we write the following expressions of the 95% confidence intervals for Q_{02}, Q_{05}, and Q_{30} when $\hat{Q}$ is known:

$$4.9\overline{Q} \leq Q_{02} \leq 7.1\overline{Q} \tag{22}$$

$$3.3\overline{Q} \leq Q_{05} \leq 4.5\overline{Q} \tag{23}$$

$$0.72\overline{Q} \leq Q_{30} \leq 1.04\overline{Q} \tag{24}$$

Accounting for uncertainty in our estimates of $\hat{Q}$, the upper and lower 95% confidence limits for each estimate can be calculated as follows, using previously calculated values of $\hat{Q}^-$ and $\hat{Q}^+$.

$$4.9\hat{Q}^- \leq Q_{02} \leq 7.1\hat{Q}^+ \tag{25}$$

$$3.3\hat{Q}^- \leq Q_{05} \leq 4.5\hat{Q}^+ \tag{26}$$

$$0.72\hat{Q}^- \leq Q_{30} \leq 1.04\hat{Q}^+ \tag{27}$$

These upper and lower limits are then plotted at the appropriate exceedance frequencies.

5. The 95% confidence intervals for the estimate of Q_{95} are calculated by first computing $Q_{95\ low}$ and $Q_{95\ high}$ by substituting $\hat{Y}_{low}$ and $\hat{Y}_{high}$, respectively, in equation 13. Then use the following equations to calculate Q^-_{95} and Q^+_{95}:

$$Q^-_{95} = Q_{95\ low} - \{0.130\ [1.07 + (9.24)(10^{-14})\hat{Y}^4_{low} - (5.69)(10^{-10}) \hat{Y}^3_{low} + (1.30)(10^{-6})\hat{Y}^2_{low} - (1.29)(10^{-3})\hat{Y}_{low}]^{1/2}\}\ A_D \tag{28}$$

$$Q_{95}^{+} = Q_{95\ high} + \{0.130\ [1.07 + (9.24)(10^{-14})\hat{Y}_{high}^{4} - (5.69)(10^{-10})$$

$$\hat{Y}_{high}^{3} + (1.30)(10^{-6})\hat{Y}_{high}^{2} - (1.29)(10^{-3})\hat{Y}_{high}]^{1/2}\}\ A_D \qquad (29)$$

RESULTS

Table 6 shows the data used to synthesize the flow-duration curves of the three test streams, and it and Figures 13-15 compare the results with the actual flow-duration data. The 95% confidence intervals for each estimate are also shown, calculated both for the case where mean basin elevation is known ($\bar{Y}$) and estimated from Y_{max} and Y_{min} ($\hat{Y}$). Each potential user must judge for himself whether a prediction method is sufficiently accurate for his purposes. However, it would appear that the method presented here is accurate enough for most purposes, and is probably as accurate as is possible using readily available information. Further indication of the accuracy of the method is given in Table 7, where estimated and actual flow-duration parameters are compared for the 21 locations analyzed in developing the method (using estimated mean basin elevation, $\hat{Y}$).

PHYSICAL BASIS FOR RESULTS

Barrows (1933), Knox and Nordenson (1955), Siccama (1968), and Engman and Hershfield (1969) have noted an increase in measured annual precipitation with elevation in northern New England. Studies of "occult" precipitation (fog drip and rime) suggest that the actual increase is even greater than that reflected in precipitation records (Vogelmann _et al._, 1968; Schlesinger and Reiners, 1974). Furthermore, there is good reason to

Table 6. Data Used in Testing Method of Estimation of Flow-Duration Curves and Results of Tests

Estimate 1 uses estimated mean basin elevation, $\hat{Y}$; estimate 2 uses measured mean basin elevation $\overline{Y}$.

							Streamflows (ft^3/s)				
River	Region	A_D (mi^2)	Y_{max} (ft)	Y_{min} (ft)		$\overline{Y}$, $\hat{Y}$ (ft)	Q_{02}	Q_{05}	$\overline{Q}$	Q_{30}	Q_{95}
Big Brook	I	6.36	3168	1680	est. 1	2060	90.1	58.5	15.0	13.2	1.71
					est. 2	2150	91.7	59.6	15.3	13.4	1.69
					obs	2150	102	59.0	15.0	12.5	2.04
Smith River	II	85.80	2920	450	est. 1	1250	776	504	129	114	7.44
					est. 2	1260	776	505	129	114	7.54
					obs	1260	866	554	142	121	12.30
Pemigewasset at Woodstock	I	193.00	5249	615	est. 1	2120	2770	1800	462	406	49.5
					est. 2	2490	2980	1940	497	438	71.8
					obs	2490	2860	1900	510	450	75.3

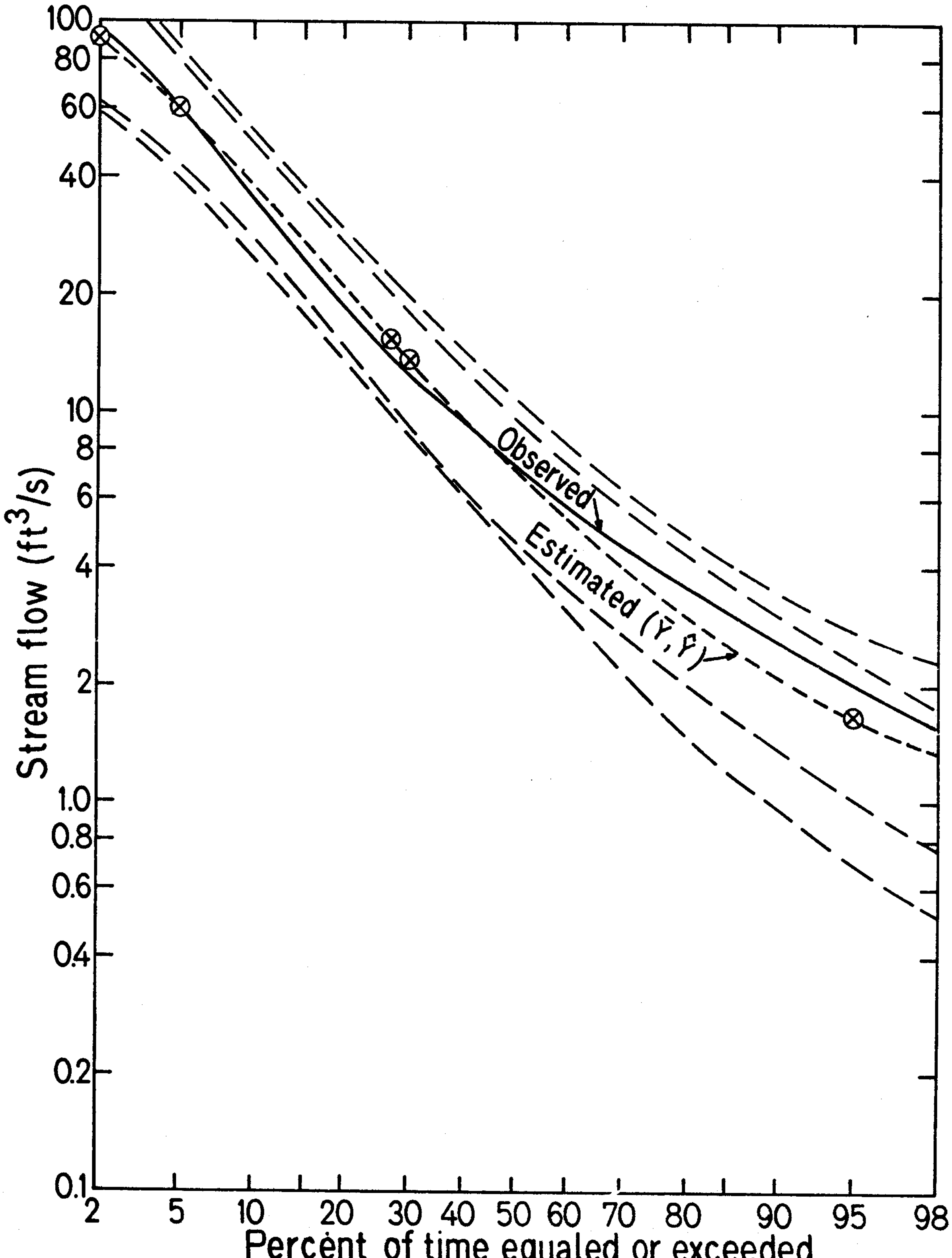

Figure 13. Comparison of Estimated and Observed Flow-Duration Curves for Big Brook Near Pittsburg, New Hampshire. Estimates Using $\bar{Y}$ (circles) and $\hat{Y}$ (x's) are Essentially Identical (see Table 6). Inner Pair of Long-Dashed Lines are 95% Confidence Bands for Estimates Based on $\bar{Y}$, Outer Pair are Bands Based on $\hat{Y}$.

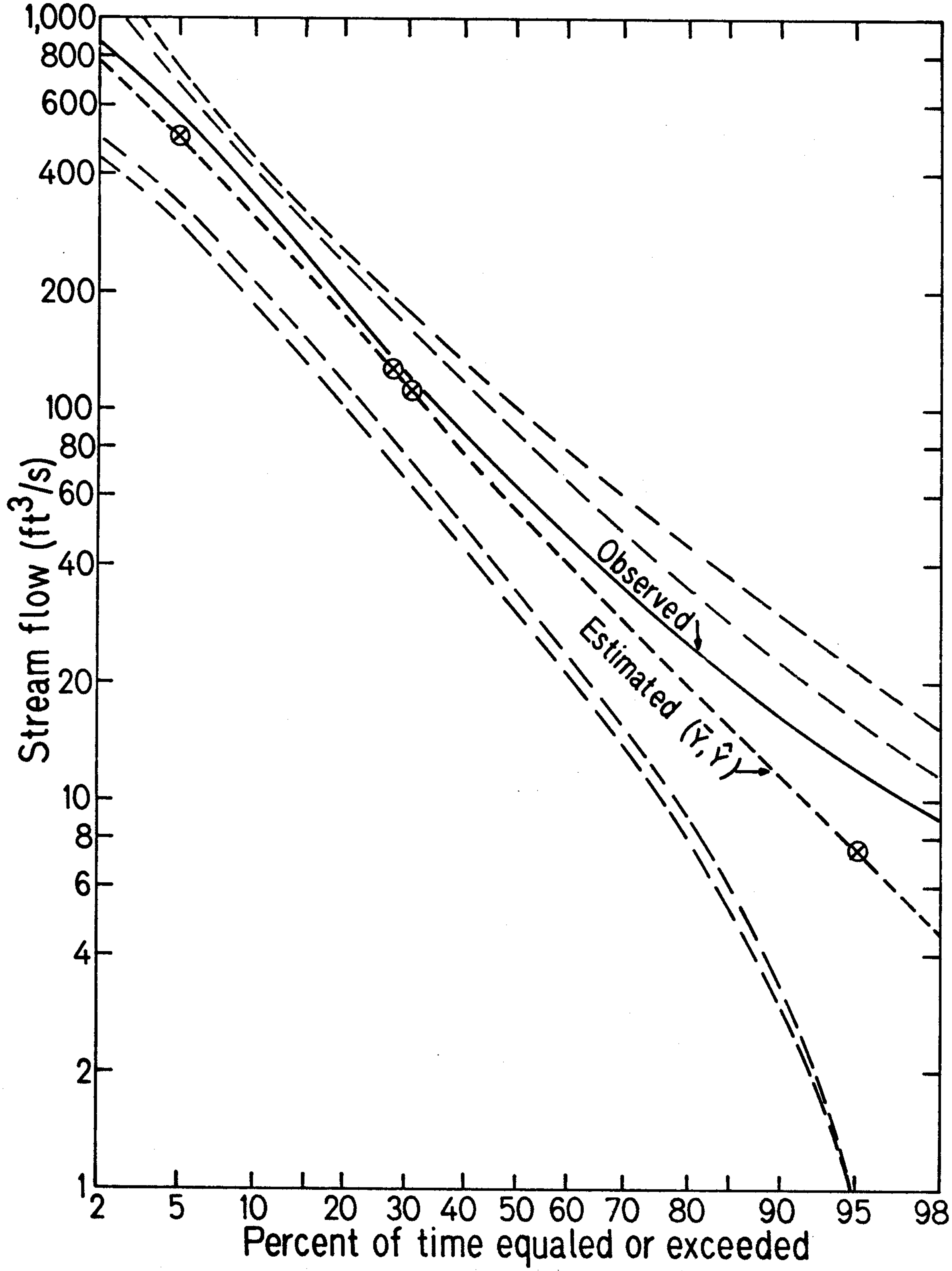

Figure 14. Comparison of Estimated and Observed Flow-Duration Curves for Smith River Near Bristol, New Hampshire. Estimates Using $\overline{Y}$ (circles) and $\hat{Y}$ (x's) are Essentially Identical (see Table 6). Inner Pair of Long-Dashed Lines are 95% Confidence Bands for Estimates Based on $\overline{Y}$, Outer Pair are Bands Based on $\hat{Y}$.

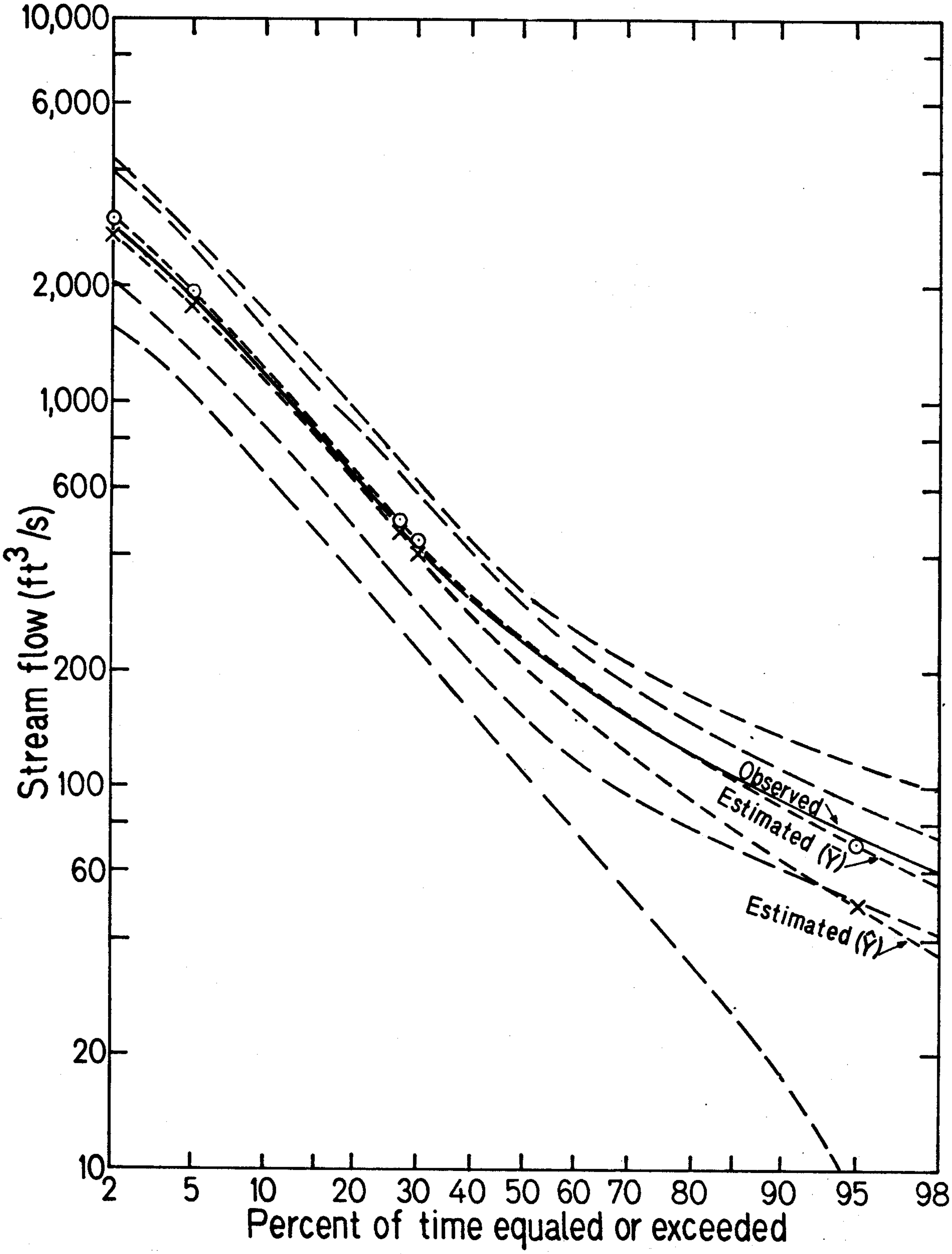

Figure 15. Comparison of Estimated and Observed Flow-Duration Curves for Pemigewasset River at Woodstock, New Hampshire. Estimates Based on $\bar{Y}$ (circles) and $\hat{Y}$ (x's) are Shown Separately (see Table 6). Inner Pair of Long-Dashed Lines are 95% Confidence Bands for Estimates Based on $\bar{Y}$, Outer Pair are Bands Based on $\hat{Y}$.

Table 7. Comparison of Estimated and Observed Flow-Duration Data for 21 Streams Used in Developing Method

Stream, Region	A_D (mi^2)		$\hat{Y}$ (ft)	Streamflows (ft^3/s)				
				Q_{02}	Q_{05}	$\overline{Q}$	Q_{30}	Q_{95}
Diamond R., I	153	est	2030	2150	1400	358	315	35.8
		obs	----	2250	1400	357	275	38.0
Wild R., I	69.5	est	2030	976	634	163	143	16.2
		obs	----	1190	670	174	125	14.5
Ellis R., I	10.9	est	3050	187	122	31.2	27.5	6.45
		obs	3340	180	120	34.3	29.3	6.45
Lucy Bk., I	4.68	est	1520	58.4	37.9	9.73	8.56	0.58
		obs	1540	62.0	40.5	10.7	8.40	1.20
Saco R., I	386	est	2320	5770	3750	961	846	122
		obs	----	5300	3450	929	770	140
Mohawk Bk., III	8.87	est	650	76.6	49.8	12.8	11.2	0.43
		obs	590	72.0	47.0	12.1	11.5	0.90
Oyster R., I	12.1	est	170	101	65.4	16.8	14.7	0.78
		obs	200	105	70.0	18.2	19.5	0.86
Dudley Bk., I	4.97	est	150	41.0	26.7	6.84	6.01	0.33
		obs	140	50.0	31.7	6.95	5.45	0.05

Table 7. (Cont.)

Stream, Region	A_D (mi^2)		$\hat{Y}$ (ft)	Streamflows (ft^3/s) Q_{02}	Q_{05}	$\overline{Q}$	Q_{30}	Q_{95}
Stevens Bk., II	2.94	est	1500	28.3	18.4	4.71	4.15	0.36
		obs	1630	34.5	20.5	4.67	3.45	0.10
Baker R. (Rumney), II	143	est	1890	1510	980	252	221	28.6
		obs	1580	1450	980	253	205	24.3
Pemigewasset R. (Plymouth), I	622	est	2010	8700	5650	1450	1280	142
		obs	1850	7400	4800	1348	1200	182
W. Br. Warner R., III	5.75	est	1450	60.0	39.1	10.0	8.82	0.66
		obs	1500	80.0	46.5	11.3	8.10	0.39
Warner R., III	146	est	1150	1420	926	237	209	11.0
		obs	970	1300	870	234	233	12.1
Soucook R., III	76.8	est	680	668	434	111	98.0	3.73
		obs	----	620	390	106	108	6.60
S. Br. Piscataquog R., III	104	est	880	951	618	158	139	5.72
		obs	----	900	600	164	163	8.20
Stony Bk. trib., I	3.60	est	1370	43.2	28.1	7.20	6.34	0.37
		obs	1390	44.0	26.5	6.80	6.40	0.27

Table 7. (Cont.)

Stream, Region	A_D (mi^2)		$\hat{Y}$ (ft)	Streamflows (ft^3/s) Q_{02}	Q_{05}	$\overline{Q}$	Q_{30}	Q_{95}
Ammonoosuc R. (B. Jct.), III	87.6	est	2840	1120	728	187	164	43.9
		obs	2510	1100	730	205	175	42.5
Mink Bk., II	4.60	est	1420	43.3	28.2	7.22	6.36	0.50
		obs	1450	42.0	26.7	6.70	6.25	0.18
Mascoma R., II	80.5	est	1610	796	517	133	117	11.4
		obs	1400	680	440	114	100	9.20
Cold R., II	82.7	est	960	689	448	115	101	4.96
		obs	----	700	445	115	102	7.20
S. Br. Ashuelot R., II	36.0	est	1480	344	274	57.4	50.5	4.25
		obs	1280	345	218	58.8	56.0	3.45

believe that average evapotranspiration decreases with elevation in the region due to negative vertical gradients of temperature, solar radiation, and length of growing season, and positive vertical gradients of relative humidity and percentage of precipitation occurring as snow (Siccama, 1974). Average streamflow is, of course, the difference between average precipitation and average evapotranspiration (neglecting deep seepage), and the strong relations between $\hat{Q}_{in}$ and $\hat{Y}$ shown in equations 12-I - 12-III are undoubtedly the result of these factors.

The fact that low flows, in particular Q_{95}/A_D, are positively correlated with elevation is more surprising. However, a plausible physical explanation for this phenomenon can be suggested. First, there is evidence that the average number of days and hours with precipitation increases with elevation in both summer and winter (Engman and Hershfield, 1969; R. L. Hendrick, unpub. data). Second, more snow falls at higher elevations, and the generally lower temperatures and higher variability of slope and aspect means that the melting of this snow is spread out over longer periods at higher elevations. The net result of these factors is that at higher elevations there is a more continuous input of liquid water into the hydrologic cycle and, during the growing season, less depletion of this water by evapotranspiration. Finally, it appears that variations in bedrock and surficial geology among basins are generally hydrologically insignificant. Thus the elevation-dependent climatic effects are reflected in the strong correlations between Q_{95}/A_D and $\hat{Y}$ shown in equation 13.

Further work on the effects of elevation and topoclimate on the water balance, streamflow variability, and floods in northern New England is currently under way by the author.

Chapter III

Section III

BASIC COMPUTER PROGRAM FOR CALCULATION OF FLOW-DURATION PARAMETERS FROM BASIN AREA AND ELEVATION DATA

S. Lawrence Dingman

Below is a listing of the program, called FDCOMP. The input data in lines 900-920 are for Big Brook, assuming mean elevation is not known and must be estimated from the highest and lowest elevations by equation 5. The printouts for this case and for the case where mean elevation is known and entered in line 920 are shown following the listing.

```
FDCOMP          14:57           06-OCT-77


1 REM CALCULATES FLOW-DURATION PARAMETERS FOR UNGAGED LOCATIONS.
2 REM
3 REM INPUT DATA ARE: 1) NAME OF STREAM AND HYDROLOGIC REGION (1,
4 REM 2, OR 3) (LINE 900); AND 2) DRAINAGE AREA (MI**2) , ELEVATION
5 REM OF HIGHEST POINT IN BASIN (FT), AND ELEVATION OF LOWEST POINT
6 REM IN BASIN (FT) (LINE 910), AND MEASURED MEAN BASIN ELEVATION
7 REM (FT) (IF UNKNOWN, ENTER 0) (LINE 920).
8 REM  OUTPUT IS MEAN BASIN ELEVATION, ESTIMATED OR ACTUAL (YMEAN, FT),
9 REM  ESTIMATED MEAN FLOW (QMEAN, FT**3/S), AND 2%, 5%, 30%, AND 95%
10 REM EXCEEDANCE FLOWS (Q02, Q05, Q30, Q95, FT**3/S). 95% CONFIDENCE
11 REM INTERVALS FOR EACH ESTIMATE ARE ALSO GIVEN.
12 REM
15 READ N1$, R
20 READ A, Y9, Y0
25 READ Y5
27 IF Y5 > 0 THEN 65
30 LET H = Y9 - Y0
40 LET Y1 = Y0 + .18*H
50 LET Y5 = Y0 + .324*H
60 LET Y8 = Y0 + .468*H
63 GO TO 70
65 LET Y1 = Y5
66 LET Y8 = Y5
70 LET B(1) = 5.13E-04
80 LET C(1) = 1.3
90 LET D(1) = 1616
100 LET E(1) = .367
```

```
110 LET F(1) = 1.01E07
120 LET B(2) = 3.95E-04
130 LET C(2) = 1.01
140 LET D(2) = 1646
150 LET E(2) = .286
160 LET F(2) = 3.77E06
170 LET B(3) = 3.81E-04
180 LET C(3) = 1.19
190 LET D(3) = 1128
200 LET E(3) = .172
210 LET F(3) = 1.52E06
220 LET Q1 = (C(R) + B(R)*Y1)*A
230 LET Q5 = (C(R) + B(R)*Y5)*A
240 LET Q8 = (C(R) + B(R)*Y8)*A
250 LET Q2 = Q1 - (E(R)*(1.06 + ((Y1 - D(R))**2)/F(R))**(1/2))*A
260 LET Q7 = Q8 + (E(R)*(1.06 + ((Y8 - D(R))**2)/F(R))**(1/2))*A
270 LET S5 = .88*Q5
280 LET S2 = .72*Q2
290 LET S7 = 1.04*Q7
300 LET T5 = 3.9*Q5
310 LET T2 = 3.3*Q2
320 LET T7 = 4.5*Q7
330 LET U5 = 6*Q5
340 LET U2 = 4.9*Q2
350 LET U7 = 7.1*Q7
360 LET V5 = (.0796 - .107E-03*Y5 + .901E-07*(Y5**2))*A
370 LET V1 = (.0796 - .107E-03*Y1 + .901E-07*(Y1**2))*A
380 LET V8 = (.0796 - .107E-03*Y8 + .901E-07*(Y8**2))*A
390 LET G1 = 1.07 + 9.24E-14*(Y1**4) - 5.69E-10*(Y1**3)
391 LET G2 = 1.3E-06*(Y1**2) - 1.29E-03*Y1
392 LET V2 = V1 - (.13*(G1 + G2)**(1/2))*A
400 LET G3 = 1.07 + 9.24E-14*(Y8**4) - 5.69E-10*(Y8**3)
401 LET G4 = 1.3E-06*(Y8**2) - 1.29E-03*Y8
402 LET V7 = V8 + (.13*(G3 + G4)**(1/2))*A
410 PRINT N1$
420 PRINT "REGION" R
430 PRINT
440 PRINT "YMEAN =" Y5
450 PRINT "Y-95 =" Y1, "Y+95 =" Y8
460 PRINT
470 PRINT "QMEAN =" Q5
480 PRINT "QM-95 =" Q2, "QM+95 =" Q7
490 PRINT
500 PRINT "Q02 =" U5
510 PRINT "Q02-95 =" U2, "Q02+95 =" U7
520 PRINT
530 PRINT "Q05 =" T5
540 PRINT "Q05-95 =" T2, "Q05+95 =" T7
550 PRINT
560 PRINT "Q30 =" S5
570 PRINT "Q30-95 =" S2, "Q30+95 =" S7
580 PRINT
590 PRINT "Q95 =" V5
600 PRINT "Q95-95 =" V2, "Q95+95 =" V7
900 DATA BIG BROOK,1
910 DATA  6.36,3168,1680
920 DATA 0
999 END
```

Mean elevation estimated from equation 5:

```
FDCOMP          14:56          06-OCT-77


BIG BROOK
REGION 1

YMEAN = 2162.11
Y-95 = 1947.84                    Y+95 = 2376.38

QMEAN = 15.3223
QM-95 = 12.2077                   QM+95 = 18.4885

Q02 = 91.9337
Q02-95 = 59.8179                  Q02+95 = 131.269

Q05 = 59.7569
Q05-95 = 40.2855                  Q05+95 = 83.1984

Q30 = 13.4836
Q30-95 = 8.78956                  Q30+95 = 19.2281

Q95 = 1.71368
Q95-95 = 0.706657                 Q95+95 = 2.7951


TIME:  0.15 SECS.
```

Mean elevation measured and entered in line 920:

```
FDCOMP          15:00          06-OCT-77


BIG BROOK
REGION 1

YMEAN = 2150
Y-95 = 2150    Y+95 = 2150

QMEAN = 15.2828
QM-95 = 12.8478                QM+95 = 17.7177

Q02 = 91.6966
Q02-95 = 62.9544               Q02+95 = 125.796

Q05 = 59.6028
Q05-95 = 42.3979               Q05+95 = 79.7296

Q30 = 13.4488
Q30-95 = 9.25045               Q30+95 = 18.4264

Q95 = 1.692
Q95-95 = 1.03826               Q95+95 = 2.34573


TIME:  0.14 SECS.
```

Chapter IV

REGIONAL FLOOD ANALYSIS

Francis R. Hall

INTRODUCTION

A useful approach to regionalization of hydrologic data can be illustrated by the index flood method of the U.S. Geological Survey (Dalrymple, 1960). The flood events to be considered are the annual instantaneous flood peaks taken from records of the U.S. Geological Survey (U.S.G.S., 1975 and references cited therein). The original method will be followed fairly closely although it has been replaced to considerable extent by computer-based statistical analyses. Nevertheless, the index flood approach has value in requiring a good deal of hydrologic reasoning and familiarity with the data. The method is not free of statistics, however, as there is a dependency on the extremal or Gumbel distribution which has proved useful for the study of extreme values such as annual floods (Haan, 1977).

SELECTING A HOMOGENEOUS REGION

The index method depends on identifying all available gaging records including crest heights and historical data if available for a hydrologically homogeneous region. Just what constitutes such a region is not clearly defined, but the Survey utilizes a statistical criterion. A simple graphical approach as presented herein is also helpful. Initially, a listing was made of all stream-gaging stations in New Hampshire with more than 10 years of record through the 1974 water year (Table 8). Then floods of selected return periods based on a log-Pearson analysis (see Chapter II) performed by the New Hampshire Department of Public Works and Highways were plotted versus drainage area on logarithmic graph paper. Because the $Q_{2.33}$ (mean annual flood) event is important in the index flood method, this graph is given as Figure 16. A plot of Q_{10}, which is also important in the method, shows a similar pattern.

An inspection of Figure 16 reveals three distinct linear trends for unregulated streams which might be thought of from left to right as mountainous, north country, and central, with a fairly distinct clustering of regulated streams to the far right. Interestingly, it does not seem to take much regulation to place a stream to the right. For present purposes, emphasis is placed on the central trend. Table 9 lists all unregulated streams that fall on the trend on both the $Q_{2.33}$ and Q_{10} graphs or on the $Q_{2.33}$ graph alone.

Strictly speaking, the Souhegan River is regulated to some extent; however, it was selected as the index station because of length of record (65 years) and close fit to the linear trend on both the $Q_{2.33}$ and Q_{10} graphs. For the final selections, the Oyster River, Dudley Brook, and the Upper Ammonoosuc River were eliminated mainly because of distance from the others. The Survey test for homogeneity will not be discussed herein, but all streams included in the final selection do meet the criterion.

Table 8. New Hampshire Gaging Stations Through 1974

U.S.G.S.[1] Station Number	Report Number	Station Name	Drainage Area mi^2	Years[3] of Record	Regulated
01052500	1	Diamond River near Wentworth Location	153	33	
01053500	2	Androscoggin River at Errol	1045	24	x
01054000	3	Androscoggin River near Gorham	1363	46	x
01064300	4	Ellis River near Jackson	10.9	11	
01064500	5	Saco River near Conway	386	45	
01065000	6	Ossipee River at Effingham Falls	330	32	x
01072500[2]	7	Salmon Falls River near South Lebanon, Maine	147	40	x
01073000	8	Oyster River near Durham	12.1	40	
01073500	9	Lamprey River near Newmarket	183	40	x
01073600	10	Dudley Brook near Exeter	4.97	12	
01074500[4]	11	East Branch Pemigewasset River near Lincoln	104	25	
01075000	12	Pemigewasset River at Woodstock	193	35	
01075500[5]	13	Baker River at Wentworth	58.8	12	
01075800	14	Stevens Brook near Wentworth	2.94	11	
01076000	15	Baker River near Rumney	143	47	
01076500	16	Pemigewasset River at Plymouth	622	71	
01077000	17	Squam River at Ashland	57.6	24	x
01078000	18	Smith River near Bristol	85.8	56	
01081000	19	Winnipesaukee River at Tilton	471	38	x
01081500	20	Merrimack River at Franklin Junction	1507	33	x
01082000	21	Contoocook River at Peterborough	68.1	29	x
01083000	22	Nubanusit Brook near Peterborough	46.9	24	x
01084000[6]	23	North Branch Contoocook River near Antrim	54.8	46	x
01084500	24	Beards Brook near Hillsboro	55.4	25	
01085000	25	Contoocook River near Henniker	368	24	x
01085500	26	Contoocook River below Hopkinton Dam, at West Hopkinton	427	11	x

[1]U.S. Geological Survey (1975). [2]Discontinued in 1969. [3]As used in log Pearson analysis by NHDPW & H.

[4]Discontinued in 1960. [5]Discontinued in 1952. [6]Discontinued in 1970.

Table 8. (Cont'd)

U.S.G.S. Station Number	Report Number	Station Name	Drainage Area mi^2	Years of Record	Regulated
01085800	27	West Branch Warner River near Bradford	5.75	12	
01086000	28	Warner River at Davisville	146	35	
01087000	29	Blackwater River at Webster	129	34	x
01088000	30	Contoocook River at Penacook	766	13	x
01089000	31	Soucook River near Concord	76.8	23	
01089500[7]	32	Suncook River at North Chichester	157	50	x
01090800	33	Piscataquog River below Everett Dam near East Weare	63.1	12	x
01091000	34	South Branch Piscataquog River near Goffstown	104	34	
01091500	35	Piscataquog River near Goffstown	202	13	x
01092000	36	Merrimack River near Goffs Falls below Manchester	3092	12	x
01093000[6]	37	Sucker Brook at Auburn	27.8	33	x
01093800	38	Stony Brook tributary near Temple	3.60	11	
01094000	39	Souhegan River at Merrimack	171	65	x
01127880	40	Big Brook near Pittsburg	6.36	11	
01128500	41	Connecticut River at First Connecticut Lake near Pittsburg	83.0	57	
01129200	42	Connecticut River below Indian Stream near Pittsburg	254	18	x
01129300	43	Halls Stream near East Hereford, Quebec, Canada	85	13	
01129500	44	Connecticut River at North Stratford	799	34	x
01130000	45	Upper Ammonoosuc River near Groveton	232	34	
01131500	46	Connecticut River near Dalton	1514	34	x
01137500	47	Ammonoosuc River at Bethlehem Junction	87.6	35	
01138000	48	Ammonoosuc River near Bath	395	39	
01140500[8]	49	Connecticut River at Orford	3100	21	x
01141800	50	Mink Brook near Etna	4.60	12	

[7] Discontinued in 1974.

[8] Discontinued in 1921.

Table 8. (Cont'd)

U.S.G.S. Station Number	Report Number	Station Name	Drainage Area mi^2	Years of Record	Regulated
01144500	51	Connecticut River at White River Junction, Vermont	4092	13	x
01145000	52	Mascoma River at West Canaan	80.5	36	
01150500	53	Mascoma River at Mascoma	153	51	x
01152500	54	Sugar River at West Claremont	269	46	x
01154500	55	Connecticut River at North Walpole	5493	14	x
01155000	56	Cold River at Drewsville	82.7	35	
01156500[7]	57	Connecticut River at Vernon, Vermont	6266	13	x
01157000	58	Ashuelot River near Gilsum	71.1	52	x
01158000	59	Ashuelot River below Surrey Mountain Dam near Keene	101	29	x
01158500[9]	60	Otter Brook near Keene	42.3	35	
01158600	61	Otter Brook below Otter Brook Dam near Keene	47.2	16	x
01160000	62	South Branch Ashuelot River at Webb near Marlborough	36.0	54	x
01161000	63	Ashuelot River at Hinsdale	420	16	x

[9]Discontinued in 1959.

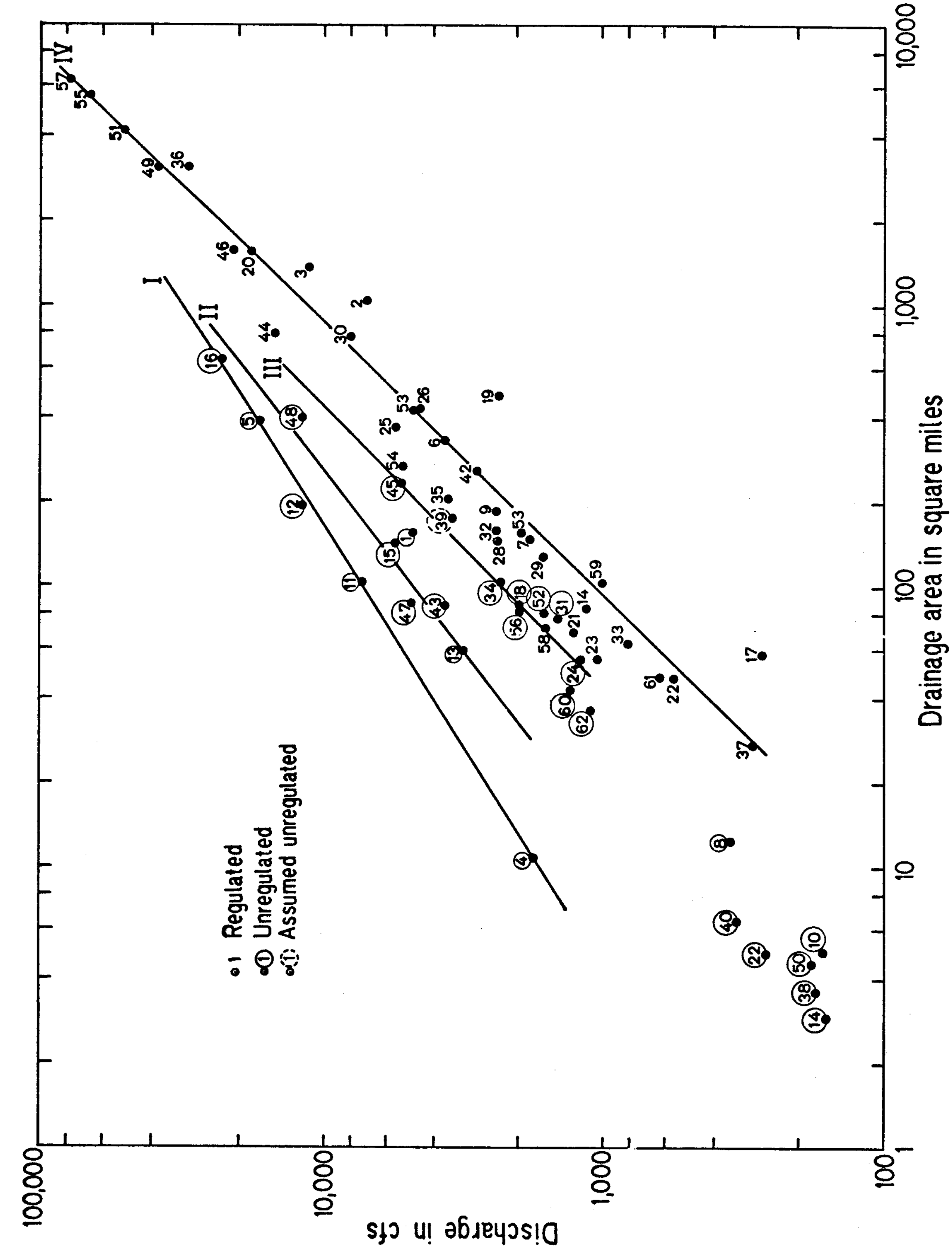

Figure 16. Mean Annual Flood ($Q_{2.33}$) Through Water Year 1974.

Table 9. Streams Showing Central Linear Trend

No.	Name	$Q_{2.33}$ and Q_{10}	$Q_{2.33}$ only	Final Selection
8	Oyster River	√		
10	Dudley Brook		√	
18	Smith River	√		√
24	Beards Brook	√		√
28	Warner River		√	
31	Soucook River	√		√
34	South Branch Piscataquog River	√		√
39	Souhegan River	√		√*
45	Upper Ammonoosuc River	√		
52	Mascoma River	√		√
56	Cold River		√	

*Index Station

DATA ANALYSIS

Flood records for each gaging station used in the analysis were compiled chronologically as illustrated for Beards Brook on Table 10. The record is dated back to 1910 although gaging did not begin until 1946 because the index station, Souhegan River, extends back to 1910. Scatter diagrams or correlation graphs were plotted for each station versus the Souhegan on a year-by-year basis for each year of actual gaging as shown for Beards Brook on Figure 17. A best fit straight line from the origin is located such that one-half of the points fall on each side. The considerable scatter of points might at first appear disconcerting; however, if all points were to fall on one line, then the two stations would be in effect measuring the same events and thus would only represent one station.

The Beards Brook record is extended by entering Figure 17 with actual Souhegan values year-by-year from 1910 to 1945 and reading off the equivalent Beards Brook values. The resulting floods are used only to assign adjusted flood magnitudes to the extended plus actual record. Return periods are calculated only for the actual period of record, and the results are subjected to a form of frequency analysis (see Chapter II) by plotting on logarithmic extremal or Gumbel paper as shown on Figure 18. For the homogeneity portion of the index flood method, not discussed herein, the $Q_{2.33}$ and Q_{10} values are taken from the graph. For the remainder of the analysis, which is described below, values are taken from the graph for selected return periods of 1.1, 1.5, 2.33, 5, 10, 20, 50, and 100 years. The Souhegan River record is analyzed directly, and the remaining streams

Table 10. Annual Floods for Beards Brook[1]

Water year	Annual Flood[2] cfs	Adj. Magnitude	Adj. T, years
1910	(1800)	15	
1911	(1110)	43	
1912	(1000)	49	
1913	(1275)	32	
1914	(1450)	24	
1915	(2375)	7	
1916	(1870)	14	
1917	(1300)	30	
1918	(715)	61	
1919	(1260)	35	
1920	(1550)	21	
1921	(2135)	10	
1922	(1565)	20	
1923	(1355)	29	
1924	(3625)	3	
1925	(800)	60	
1926	(1020)	48	
1927	(1125)	42	
1928	(2440)	6	
1929	(890)	56	
1930	(990)	50	
1931	(1375)	26	
1932	(1385)	25	
1933	(1265)	33	
1934	(2975)	5	
1935	(1285)	31	
1936	(6800)	1	
1937	(1355)	28	
1938	(4250)	2	
1939	(885)	58	
1940	(1675)	16	
1941	(925)	53	
1942	(1525)	22	
1943	(675)	62	
1944	(3100)	4	
1945	(950)	52	

[1]U.S. Geological Survey (1975).

[2]Parentheses mean interpolated by graphical correlation.

Table 10. (Cont'd)

Beards Brook

Water Year	cfs	Original Record[3] Magnitude	Adj. Magnitude	Adj. T, years
1946	954	19	51	1.29
1947	1140	15	41	1.61
1948	1880	4	13	5.08
1949	908	20	54	1.22
1950	1100	17	46	1.43
1951	2070	3	12	5.50
1952	1660	5	18	3.67
1953	1650	6	19	3.47
1954	1220	10	36	1.83
1955	894	21	55	1.20
1956	1500	7	23	2.87
1957	600	24	64	1.03
1958	1180	12	38	1.74
1959	1100	16	45	1.47
1960	2190	1	8	8.25
1961	885	22	57	1.12
1962	1140	13	39	1.69
1963	1020	18	47	1.40
1964	1140	14	40	1.65
1965	500	25	65	1.01
1966	644	23	63	1.05
1967	1370	8	27	2.44
1968	1210	11	37	1.78
1969	2170	2	9	7.33
1970	1260	9	34	1.94
1971	850		59	1.12
1972	1100		44	1.50
1973	1660		17	3.88
1974	2080		11	6.00

[3] 1946-1970

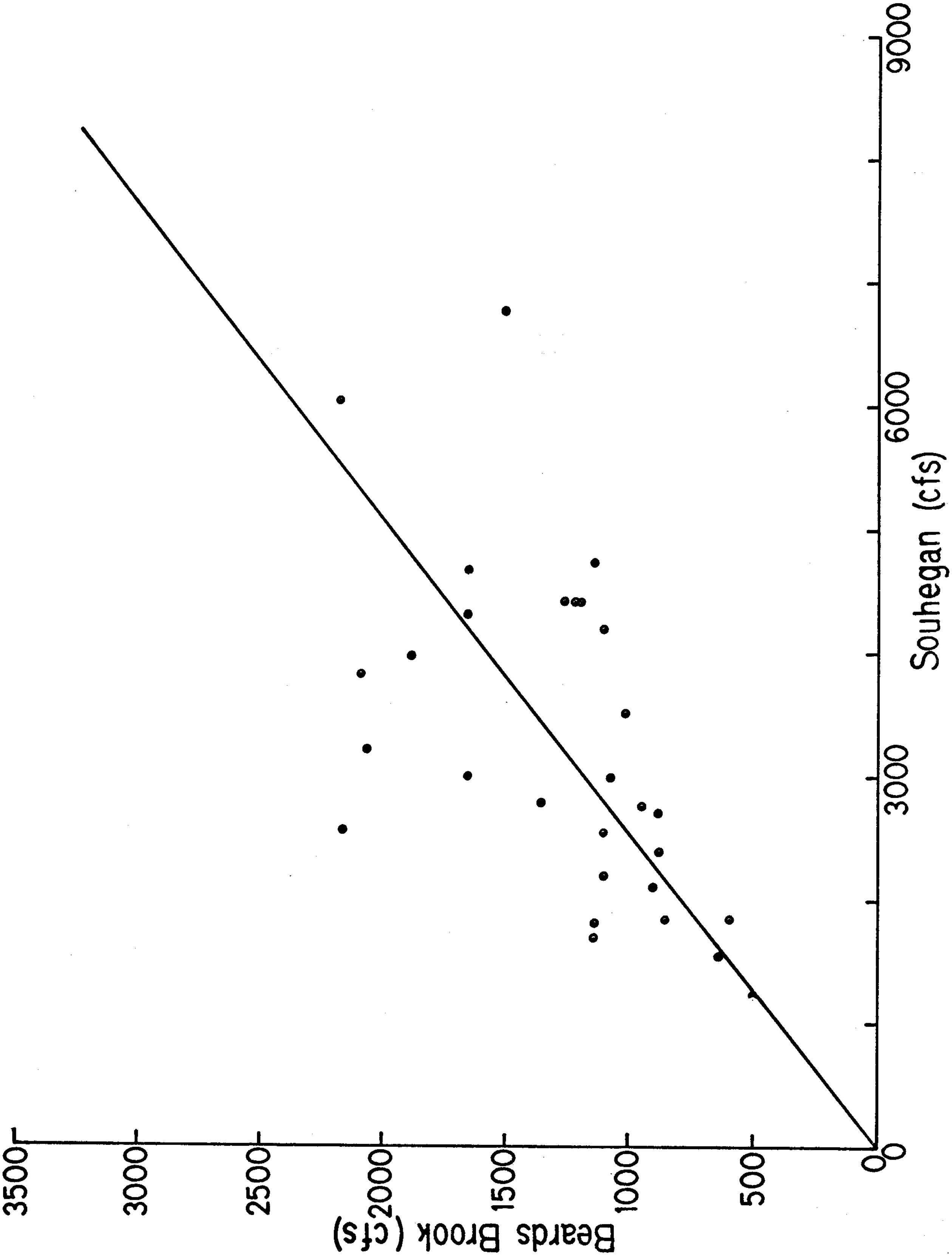

Figure 17. Correlation Graph for Beards Brook Versus Souhegan River.

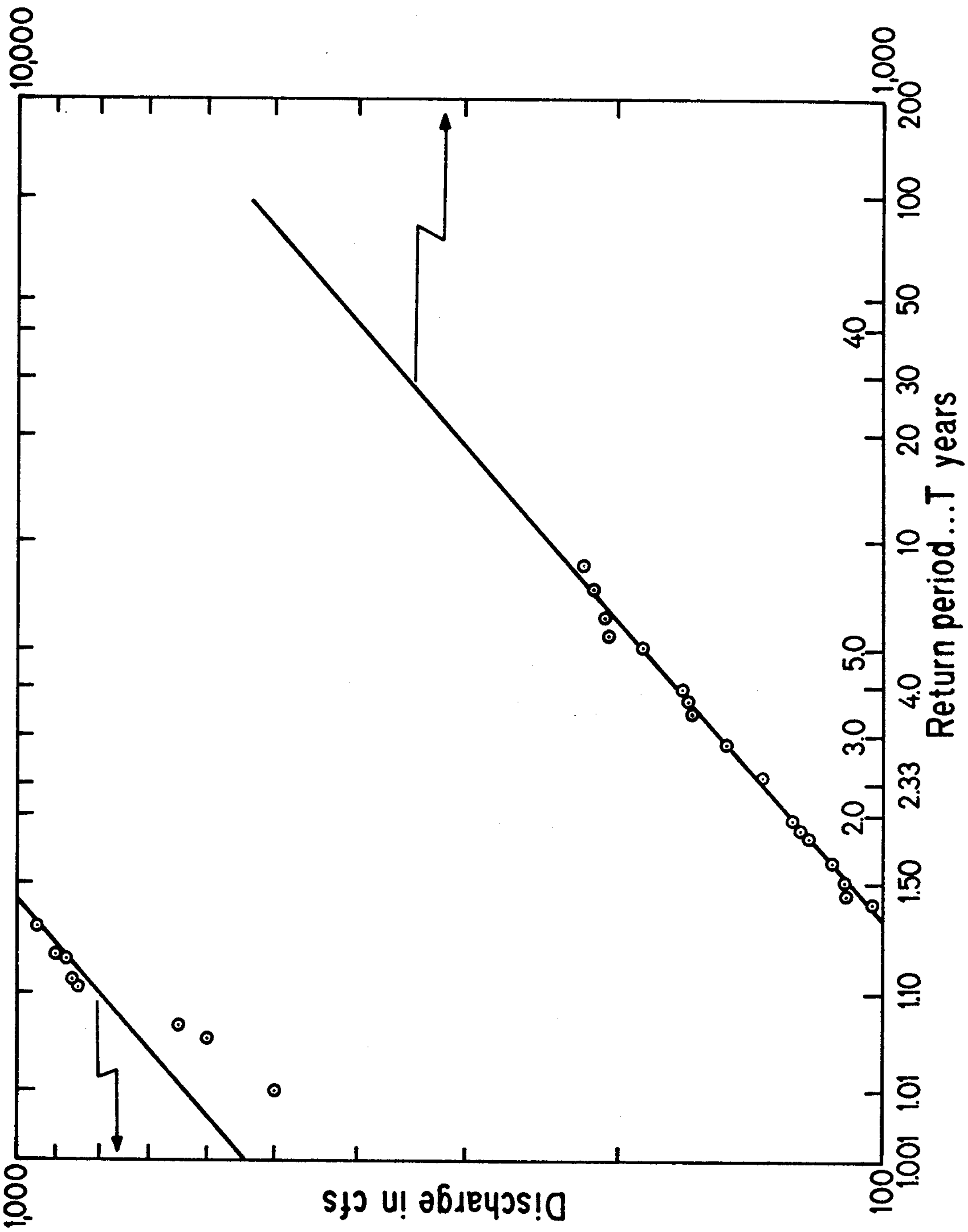

Figure 18. Frequency Graph for Beards Brook on Extremal Paper.

are handled in the same fashion as was Beards Brook.

At this point an interesting if not downright puzzling problem arises. Namely, an inspection of the Beards Brook data in Table 10 and Figure 18 indicates that the highest adjusted return period is 8.25 years, whereas the original record has one of 30 years. In fact, according to the Survey method, the adjusted period of record for Beards Brook is 35 years. A similar problem arises for all the other extended records. Therefore, in a sense the records have not been extended, but instead have shrunk! What has happened is that too many low magnitude-high return period events on the Souhegan River occurred before 1945. In fact, the first five events occurred between 1924 and 1944.

This problem of apparent loss of record calls into question the basic assumption that annual floods are random events drawn from the same population. In fact, there would appear to be either nonrandomness (meteorologic effects) or more than one population (say floods from heavy summer rainfall, hurricanes in the fall, and snow melt in the spring). Further discussion is beyond the scope of this report, but the problem should be noted. In order to complete the analysis, therefore, the records have been graphically extended by a best eye fit straight line on log extremal paper (Figure 18).

The analysis is completed in two steps as follows. First, the flood discharges for the selected return periods are taken from the graphs for each stream. Finally, the median ratio is obtained for each return period as shown in Table 11, and the results are plotted as the regional flood frequency curve on Figure 19. The second step is to prepare a graph of

Table 11. Index Flood Ratios and Other Data

No.	Stream	Area mi^2	Discharge for Indicated Return Period, Q_T: 1.1	1.5	2.33	5	10	20	50	100
18	Smith River	85.8	940	1390	1800	2600	3450	4600	6600	8600
	Ratio $Q_T/Q_{2.33}$		.52	.77	1.00	1.44	1.92	2.56	3.67	4.78
24	Beards Brook	55.4	870	1090	1360	1875	2400	3100	4250	5400
	Ratio $Q_T/Q_{2.33}$		.60	.80	1.00	1.38	1.76	2.28	3.12	3.97
28	Warner River	146	1280	1740	2290	3340	4560	6190	9000	12,000
	Ratio $Q_T/Q_{2.33}$		.56	.76	1.00	1.46	1.99	2.70	3.93	5.24
31	Soucook River	76.8	760	1090	1410	2000	2690	3550	5100	6700
	Ratio $Q_T/Q_{2.33}$		.54	.77	1.00	1.42	1.91	2.52	3.62	4.75
34	South Branch Piscataquog River	104	1250	1700	2250	3300	4480	6090	8900	11,500
	Ratio $Q_T/Q_{2.33}$		.56	.76	1.00	1.47	1.99	2.71	3.96	5.11
39	Souhegan River	171	1950	2630	3460	5020	6880	9250	13,250	17,500
	Ratio $Q_T/Q_{2.33}$		.56	.76	1.00	1.45	1.99	2.67	3.83	5.06
52	Mascoma River	80.5	830	1275	1625	2300	3100	4050	5800	7500
	Ratio $Q_T/Q_{2.33}$		.51	.78	1.00	1.42	1.91	2.49	3.57	4.62
	Median for $Q_T/Q_{2.33}$		.56	.77	1.00	1.44	1.92	2.56	3.67	4.78

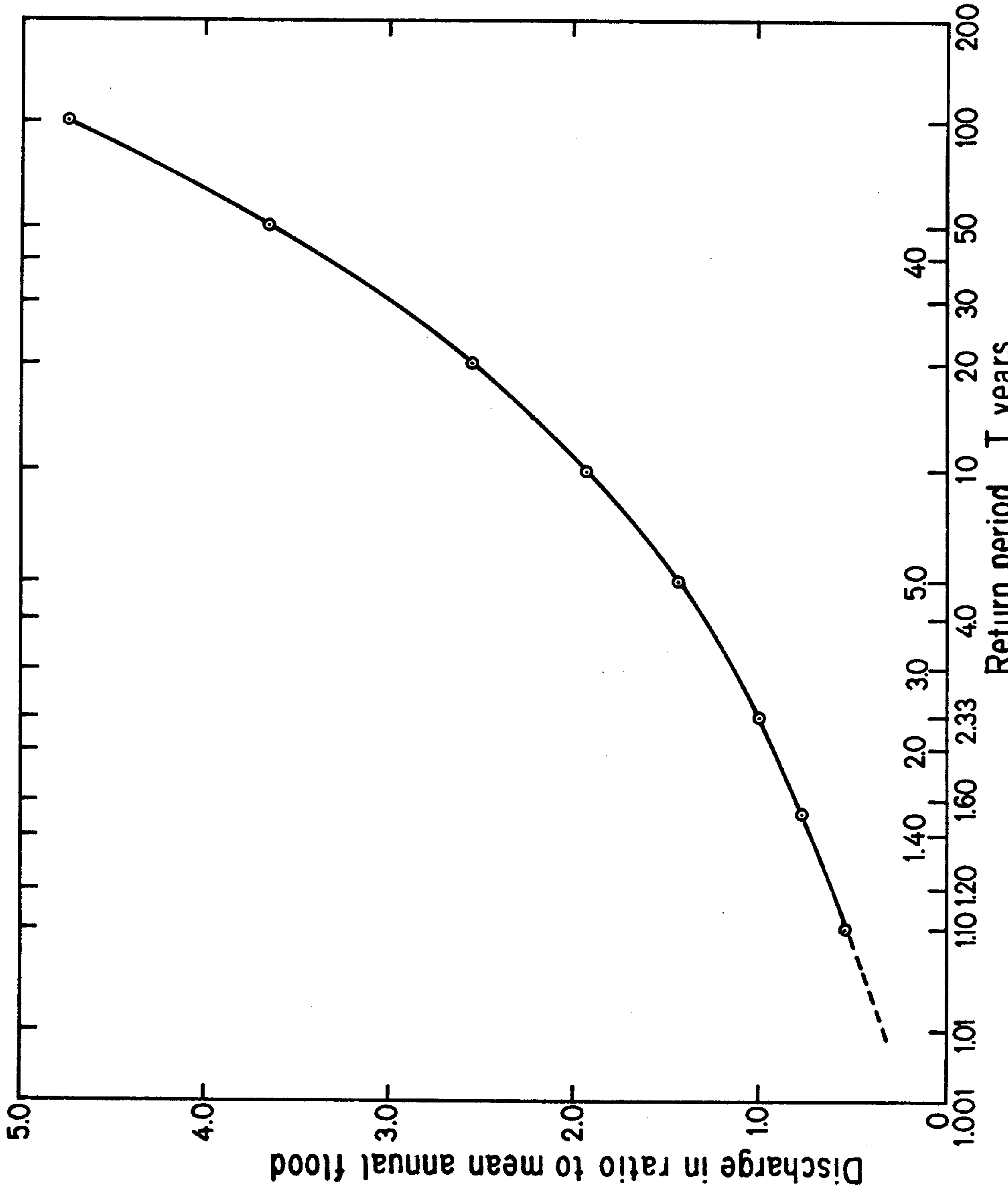

Figure 19. Regional Flood Frequency Curve.

mean annual flood versus drainage area for each station (Figure 20). Now, a dimensionless flood ratio for any desired return period up to 100 years can be obtained from Figure 19 and applied to any desired drainage area from about 30 to 200 square miles. The regional frequency curve should be approximately valid for the part of New Hampshire lying close to and west of the Merrimack River and extending from about the Massachusetts border north to Lebanon and Bristol. However, fairly mountainous areas probably should be excluded.

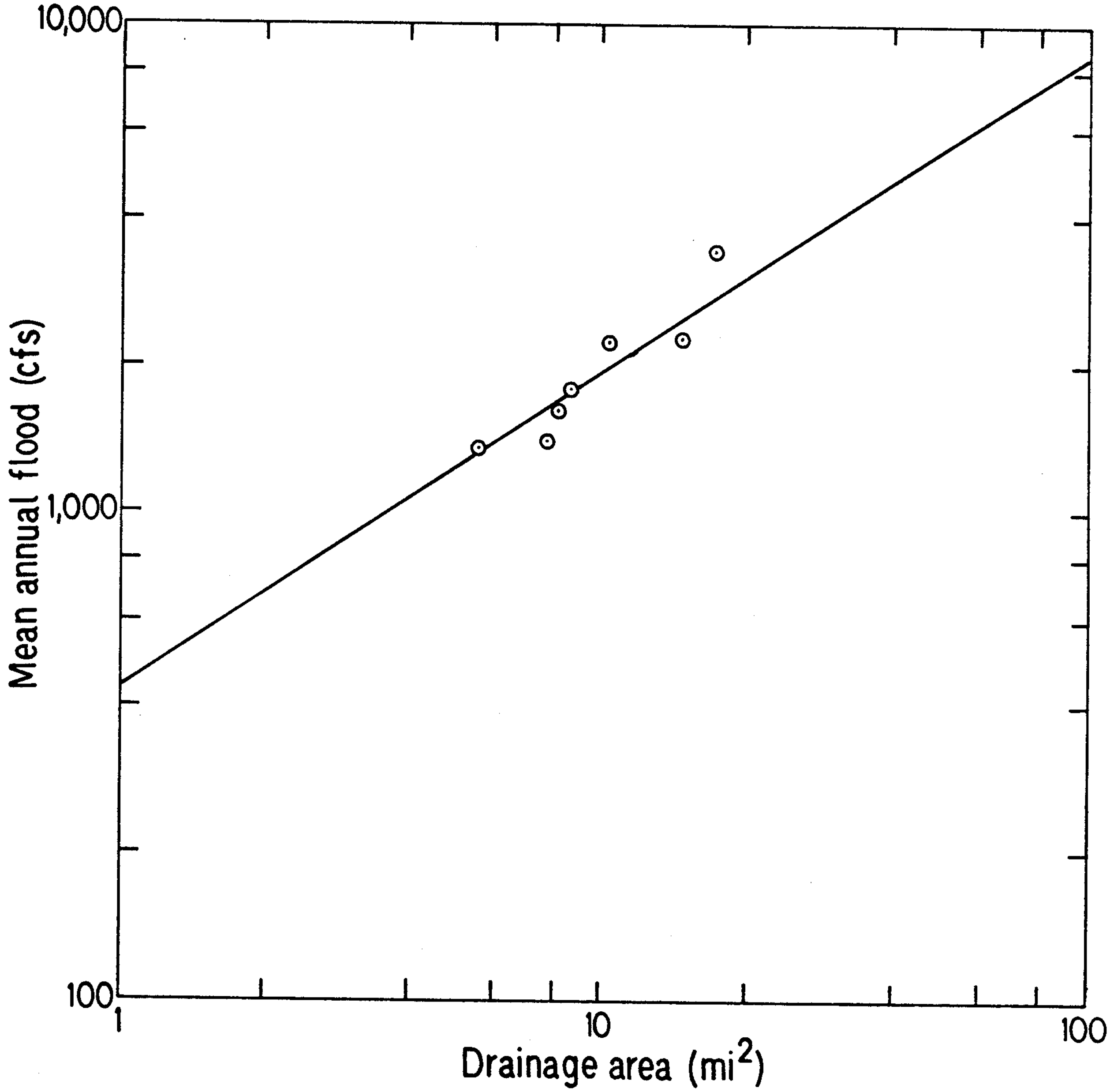

Figure 20. Mean Annual Flood Versus Drainage Area.

Chapter V

REGRESSION ANALYSIS APPROACH
Francis R. Hall

INTRODUCTION

A form of statistical analysis that is commonly referred to as regression offers a potentially valuable tool for the interpretation and utilization of hydrologic data, particularly on a regional scale (an example is given in Chapter III). The background and methodology required for regression analysis are covered well by Draper and Smith (1966), and more specific applications in hydrology are discussed by Chow (1964), Riggs (1968), and Haan (1977).

The simplest form, called linear regression, involves a relationship of general type:

$$y = a + bx \tag{30}$$

where y, dependent variable

x, independent variable

a, regression constant or intercept value of y at x = 0

b, regression coefficient or slope of x-y line

Such a relationship is obtained from calculations based on the criterion that the sum of squares of the differences between measured and calculated "y" be a minimum. These differences are commonly referred to as residuals. Goodness of fit can be evaluated in various ways, but for present purposes the standard error of estimate is quite useful. The standard error, or S.E. as it will henceforth be called, "...is the standard deviation of the distribution (assumed normal) of residuals about the regression line." (Riggs, 1968).

In many cases, a good relationship of the form of equation 30 is not obtained between measured items; however, things may be improved by applying a transformation of some type to one or both members of the data set. A logarithmic transformation is often useful for hydrologic data, and the emphasis herein will be on the situation where all variables are so transformed. Such a transformation has the added advantage that equal variance about the regression line is achieved throughout the data range as is required for regression (Riggs, 1968). Equation 30 now takes the form:

$$y = aX^{b} \tag{31a}$$

or

$$\log y = \log a + b \log x \tag{31b}$$

where log y, transformed dependent variable

log x, transformed independent variable

log a, regression constant or intercept value of y at x = 1

b, regression coefficient or slope of log x-log y line

log, logarithm to base 10

A dependent variable of interest such as stream discharge may be a function of more than one independent variable. Where this is the case, linear regression can be expanded to a more general multiple regression equation of form:

$$y = aX_1{}^{b}X_2{}^{c}X_3{}^{d}\text{---} \tag{32a}$$

or $\log Y = \log a + b \log X_1 + c \log X_2 + d \log X_3$ --- (32b)

where Y, dependent variable

X_1, X_2, X_3, ---, independent variable

a, constant

b, c, d, ---, regression coefficients

Note that the constant a, b, c, d, ---, are no longer so easily interpreted as in the simple linear regression case.

In multiple regression, there is a tendency for statistical indicators to improve as the number of independent variables is increased. Also, some of the independent variables may be correlated and thus interact with each other. In some cases, an independent variable may in effect serve as a proxy for something not included. For these reasons, the terms and results of multiple regression should be carefully evaluated, and as a rule-of-thumb, it seems advisable to reduce the number of terms as far as possible as long as the standard error remains acceptable. A comparable approach is to require that the regression coefficients be significant at some prescribed level.

When the standard error is calculated for regressions of the form of equations 31 or 32, the results will be in log units which are not easily interpreted. A useful approach is to convert the S.E. in log units to a S.E. in the form of a constant percentage about the regression line. For example, if the standard error is 0.05 log unit the following procedure is followed (Riggs, 1968):

1. Compute the antilogs at 1 + S.E. and 1 - S.E. which are ratios

to 10 (the base of the log is 10) or

1 + S.E. = 1.05 Antilog 11.22

1 - S.E. = 0.95 Antilog 8.91

2. Then the percentage errors are calculated as

   100 (11.22 - 10)/10 = 12.2 percent

   100 (10 - 8.91)/10 = -10.9 percent

It is often convenient to average these values to obtain one representative value or (12.2 + 10.9)/2 = 11.6 percent or 12 percent. It should be noted that this average value becomes less representative of the S.E. as the log unit increases.

The specific details and requirements for regression are covered well in the literature cited. Nevertheless, a few observations and precautions should be given before proceeding to the examples. The residuals from regression (measured "Y" minus computed "Y") are assumed to be independent, have a constant mean, and follow a normal distribution (Draper and Smith, 1966). Therefore, the residuals from a regression should be carefully examined for any sort of bias, and Draper and Smith offer particularly good criteria for doing so. Confidence limits can be set and t- and F-tests performed, but the results should be treated cautiously as not all requirements may be met even if the regression seems fairly good. In addition to the standard error of estimate (about the regression line), a standard error of individual prediction can also be computed. The latter will be larger than the former, and this may need to be taken into account when deciding how well the regression equation does.

Another feature to be considered is that correlation analysis and regression analysis are similar but not identical. In fact, the requirements for correlation are more stringent than for regression and according to Riggs (1968) the former may not commonly be met for hydrologic data. Also, because hydrologic data tend to be spread over several orders of magnitude or more, the correlation coefficient which depends in part on the range of data may be deceptively high when other statistics such as the standard error are not particularly good. For these reasons, the correlation coefficient should be interpreted with considerable care when computed for hydrologic data.

EXAMPLES

The following examples are taken mainly from reports of the U.S. Geological Survey. This approach is followed because the Survey has expended considerable effort on this type of analysis and because the results are readily available. Also, independent analyses performed as part of the present study did not improve on the Survey's results.

Mean Flow

A commonly sought value is the mean flow for an ungaged area for a given time period such as annual, seasonal, monthly, weekly, or daily. Usually annual flow can be obtained quite well and seasonal and monthly flows can be obtained reasonably well, but serial correlation begins increasingly to influence periods of less than a year and comes to dominate flow period of less than a month. Therefore, regression analysis is generally not suitable for these shorter time periods. Time of year also enters in because winter is characterized by snowpack accumulation and low evapotranspiration, spring is a period of snow melt and increasing evapotranspiration, summer is dominated by evapotranspiration, and fall is a period of decreasing evapotranspiration. Note that precipitation does not change much from month to month in New England. Also, the change from one season to another is not clearly defined and need not take place at the same time each year.

A detailed regression analysis has been made for 135 natural, essentially unregulated, streams in central New England (Johnson, 1970). Drainage basin areas range in size from 1.64 to 9661 square miles with most in the range of 20 to 1000. The following drainage basin characteristics were included: drainage basin area, a representative channel slope, a representative channel length, a surface-water storage factor, mean basin elevation, a forest cover factor, mean annual precipitation, a rainfall intensity factor, minimum January temperature, annual snowfall, and a soils index (related to infiltration). The complete equation for mean annual flow (not included herein) has an S.E. of 7.4 percent. Channel length has no effect on the regression, whereas eliminating the soils index drops the S.E. to 7.1 percent. The successive elimination of snow, rainfall intensity, forest cover, channel slope, and surface-water storage have no affect on the S.E., so the simplest regression equation is:

$$QA = 0.395A^{1.02}P^{0.63}E^{-0.042}T^{-0.12}, \text{ S.E.} = 7.1 \text{ percent} \qquad (33)$$

where QA, mean annual discharge in cubic feet per second

A, drainage area in square miles

P, mean annual precipitation in inches (minus 30)

E, mean basin elevation in thousands of feet above mean sea level

T, minimum January temperature in degrees Fahrenheit

Further work with equation 33 shows that the dropping of E only increases the S.E. to 7.4 percent, and the elimination of T increases the S.E. to 8.7 percent which is still quite reasonable. The elimination of P, however, raises the S.E. to 14.8 percent, and also illustrates the reasons for the precautionary comments about correlation coefficients. For example, a similar regression analysis for New Hampshire streams yields:

$$QA = 1.64A^{1.01} \tag{34}$$

with a correlation coefficient of 0.995. As shown by the results for elimination of P above, however, the S.E. is probably unacceptably large. Furthermore, an examination of the residuals from equation 34 shows a pronounced geographic basis with the equation consistently underestimating annual discharge in the northern part of New Hampshire and consistently overestimating it in southern portions. Therefore, equation 34 is not satisfactory in spite of the impressive correlation coefficient.

A regression analysis has been performed for mean monthly discharge (by the month) (Johnson, 1970). The complete results are not given herein, but a few comments will illustrate interesting features. The S.E. ranged from 9.4 percent in December to 29.0 percent in August with mean of 16.8 percent for the 12 months. The resulting equations are not as satisfactory as those for annual discharge. Drainage area and precipitation remain important in regression, but there seems almost no really consistent interaction with any of the other factors except possibly minimum January temperature and snow where there is an interesting sign reversal in April.

Low Flow

The low flow of a stream is of interest to people concerned with water supply, fish and other aquatic life, water birds, and so on. Therefore, it is unfortunate that regression analysis has not been very successfully applied in developing suitable relationships. The reasons would appear to be due mainly to the fact that, so far, it has not been possible to assign quantitative values to geologic and hydrologic factors such as storage, fracture patterns in rock, evapotranspiration losses along streams, direct evaporation from the water table, and so on.

Regression analysis for central New England streams for seven-day low flows for return periods of two years, 10 years, and 20 years gives S.E. of 55.9 percent, 92.2 percent, and 135.4 percent, respectively. Drainage area and annual precipitation are important, but other factors do not seem to be involved in a consistent way.

High Flow

The applicability of regression analysis to flood flows is illustrated well from recent work in Massachusetts by the U.S. Geological Survey (Wandle, 1977). Instantaneous flood peaks for selected return periods as determined by log-Pearson analysis (see Chapter II) were regressed on drainage basin characteristics of the types discussed under mean flow. Two additional basin characteristics were included for a shape factor and a timing factor. Data were used from 113 stream gages on natural streams with at least 10 years of record and with drainage areas ranging from 0.25 to 497 square miles. Reasonable equations were obtained on a state-wide basis with S.E. in the range of 40 to 74 percent. The equations with minor exception, however, involved four independent variables which makes them somewhat unwieldy.

Better results were obtained by dividing the State into eastern and western portions along the drainage divide between coastal and western basins. This implicitly took care of topographic and annual precipitation variations. The S.E. were not greatly improved, but the equations were simpler. Some examples are:

Eastern Massachusetts	Approximate S.E. in Percent	
$Q10 = 27.02\ A^{0.818}\ S1^{0.257}$	42	(35a)
$Q50 = 44.31\ A^{0.810}\ S1^{0.269}$	52	(35b)
$Q100 = 53.86\ A^{0.807}\ S1^{0.272}$	57	(35c)

Western Massachusetts

$Q10 = 0.017\ A^{0.909}\ St^{-0.298}\ P^{6.25}$	41	(36a)
$Q50 = 0.034\ A^{0.926}\ St^{-0.310} P^{6.09}$	52	(36b)
$Q100 = 0.046\ A^{0.937}\ St^{-0.314}\ P^{6.00}$	56	(36c)

where QT, instantaneous peak discharge in cubic feet per second for indicated return period, T

A, drainage area in square miles

S1, main channel slope in feet per mile

St, area of lakes and ponds as a percentage plus 0.5

P, mean annual precipitation in feet.

The results are not applicable to Cape Cod, Martha's Vineyard, Nantucket, or eastern Plymouth County mainly because of the thicker glacial deposits present in these areas. The 113 streams used in the study included some in New Hampshire and Vermont. Therefore, the eastern Massachusetts equations 35a-35c should be applicable to southeastern New Hampshire and a central portion north, perhaps to Manchester. The western Massachusetts equations 36a-36c are probably applicable to western New Hampshire for 10 to 20 miles north of the border.

Some interesting features of the equations are that for eastern Massachusetts, drainage area decreases and channel slope increases in relative importance as return period, and thus flood size, increases. Whereas for western Massachusetts both drainage area and area of lakes and ponds increase and annual precipitation decreases in relative importance.

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